PRIMARY AND SECONDARY COMPRESSION BEHAVIOR OF SOFT CLAYS

A THESIS SUBMITTED IN

PARTIAL FULFILLMENT OF THE REQUIREMENT FOR THE

AWARD OF THE DEGREE

OF

MASTER OF TECHNOLOGY (RESEARCH)

IN

CIVIL ENGINEERING

(GEOTECHNICAL ENGINEERING)

PRAGYAN PARAMITA DAS

DEPARTMENT OF CIVIL ENGINEERING

NATIONAL INSTITUTE OF TECHNOLOGY

ROURKELA, ODISHA- 769008

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CERTIFICATE

This is to certify that the project entitled "**Primary and secondary compression behavior of soft clays"** submitted by Miss. Pragyan Paramita Das. (Roll No. 613CE3003) in partial fulfillment of the requirements for the award of Master of Technology (Research) Degree in Civil Engineering at NIT Rourkela is an authentic work carried out by her under my supervision and guidance.

To the best of my knowledge, the matter embodied in this report has not been submitted to any other university/institute for the award of any degree or diploma.

Place: Rourkela

Date:

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Pragyan Paramita Das

ABSTRACT

During the construction of civil engineering structures on soft clayey deposits, engineers must have to consider the consolidation behavior of the deposit. Due to the natural viscosity of clayey soils, their consolidation behaviour is strongly influenced by vertical stress as well as by organic content. As more and more constructions are concentrated in densely populated urban areas, there is an increasing need to construct buildings and geotechnical structures on soft clay materials, which usually produce significant creep deformation. Although a lot of research work has been carried out related to the secondary consolidation behaviour of a natural clay material, there are still many questions about this phenomenon. Detailed laboratory studies on soft soil are necessary for the better understanding and, consequently, better prognosis of this behavior. Some of these studies were performed and are presented in this project. One dimensional consolidation tests were conducted on ten numbers of plastic clays of plasticity index (I_p) ranging from 13% to 102%. The samples were prepared by mixing different percentages of silt with commercially available Na and Ca bentonite to have representative samples of different plasticity. Six incremental consolidation pressures were given on each soil sample of different plasticity index starting from 50 kPa, 100 kPa, 200 kPa, 400 kPa, 800 kPa and 1600 kPa. A detailed study of the results obtained from incremental loading test was investigated. From the consolidation test data, the values of different parameters like primary compression index (C_c), secondary compression index (C_α), time rate of consolidation (C_v), coefficient of volume compressibility (m_v), permeability (k) were determined. The parameters were studied for factors like stress range, plasticity index and void ratio.

It was found that C_c values increases with increase in the plasticity of the soil. The present experimental values of C_c were compared with those values by using available empirical relations. Correlation was established between C_c and I_p. C_α decreases with increase in stress range but increases with increase in plasticity index. For low to medium plastic clays, C_α is independent of consolidation pressure but for highly plastic clay, C_α is stress dependent. Correlation was established between C_α, I_p, and consolidation pressure ($\sigma^\prime$). Correlation was also established between $C_\alpha/(1+e_p)$ with I_p. Other consolidation parameters like coefficient of consolidation (C_v), coefficient of volume compressibility (m_v) and permeability (k) were studied with respect to plasticity index (I_p) and consolidation pressure ($\sigma^\prime$). Coefficient of consolidation (C_v) is less dependent on stress for highly plastic soil. Coefficient of volume

compressibility (m_v) is high for soft clays at low stress level. Permeability (k) is inversely proportional to stress level.

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Symbols and Notations

C_c = Primary compression index

C_α = Coefficient of secondary consolidation

C_v = Coefficient of consolidation

e_f = Final void ratio

e_p = Void ratio at the end of primary consolidation

I_p = Plasticity index

k = Coefficient of Permeability

m_v = Coefficient of volume compressibility

t = Time

w_l = liquid limit

w_p = plastic limit

σ' = Effective vertical stress

Chapter-1

INTRODUCTION

Chapter 1

Introduction

1.1 General

Growing population and acute shortage of land has led to land reclamation in many developed countries. For convenience of sea transport, many big cities in the world are located on clayey deposit around coastal area, thus large number of structures are built on soft soils. For civil engineering projects related to soft clayey deposits, engineers inevitably have to consider the consolidation behaviour of the deposit. During the construction of these structures, the soil layer is subjected to a compressive stress, and it will exhibit a certain amount of compression. This compression is achieved through a number of ways, including rearrangement of the soil solids and / or extrusion of the pore air and/or water.

Consolidation generally is a process by which soils decrease in volume by squeezing out pore water on account of gradual dissipation of excess hydrostatic pressure induced by an imposed total stress. According to Karl Terzaghi "consolidation is the process which involves decrease in water content of a saturated soil without replacement of water by air." Terzaghi shown the consolidation process in a clay soil subjected to loading is analogous to the behaviour of the spring – piston model. Springs represent soil skeleton. Spring surrounded by water represents saturated soil. Perforations in the pistons are analogous to the voids that impart permeability to soil.

When stress is removed from a consolidated soil, the soil will rebound, regaining some of the volume it had lost in the consolidation process. If the stress is reapplied, the soil will consolidate again along a recompression curve, defined by the recompression index. The soil which had its load removed is considered to be overconsolidated. The highest stress that it has been subjected to is termed the preconsolidation stress. The over consolidation ratio or

OCR is defined as the highest stress experienced divided by the current stress. A soil which is currently experiencing its highest stress is said to be normally consolidated and to have an OCR of one. A soil could be considered under-consolidated immediately after a new load is applied but before the excess pore water pressure has had time to dissipate.

In geotechnical practice we generally see that one-dimensional consolidation of soils is conveniently divided into three portions, viz., (a) initial compression, (b) primary consolidation, and (c) secondary consolidation. Such a classification involves the clear implication that the three effects proceed in that order.

After primary consolidation comes the phenomenon of secondary compression. Once all the water is squeezed out, it might be expected that settlement would stop. Interestingly, that is not the case. Settlement continues even after all the excess pore water is squeezed out. This phenomenon is known as secondary compression, is due to the fact that soil particles start to rearrange their orientation into a more stable configuration. Rearrangement of soil particles thus causes further settlement of the foundation. So, in the design of foundations on soft ground, the long-term settlement of the subsoil must be calculated in order to predict the post-construction performance of the structure.

Clayey soils are extremely complex natural materials containing a significant amount of finely dispersed clayey particles less than 0.002 mm in diameter, which have the greatest influence on the difficult physical, mechanical and physicochemical processes inside these materials. Because of the difficult nature of these material there are a lot of aspects influencing their secondary consolidation behaviour such as composition (i.e. content of the clayey particles), stress history, change in temperature, biochemical environment and transformations.

1.2 Background and introduction to the problem

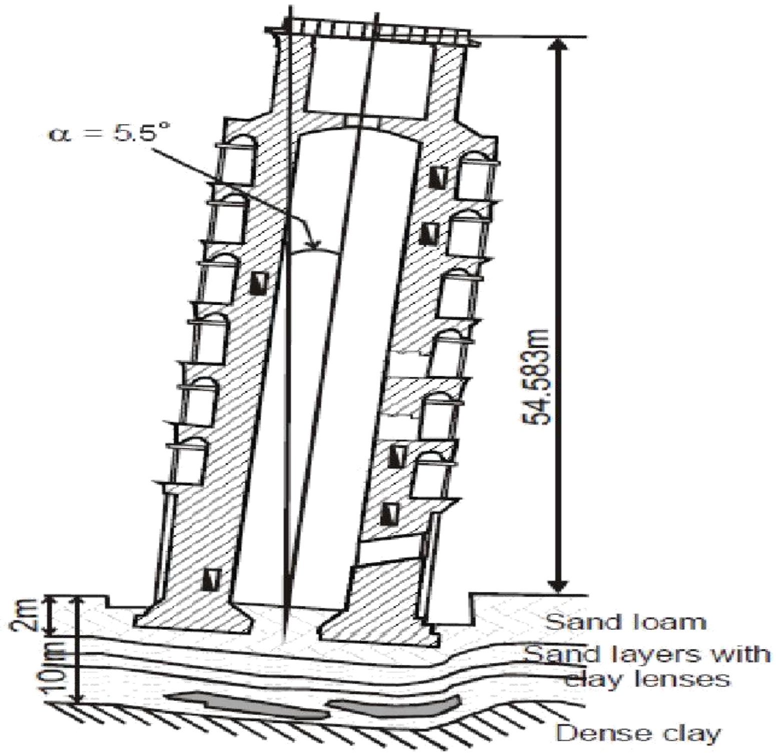
In comparison with sandy material, clayey soils usually display large deformations seen for instance in the form of prolonged settlements, tilts and horizontal shifts of buildings and geotechnical structures, or slow slippage of the natural slopes and embankments. Based on the visual observation of deformations of ancient structures and natural slopes, the existence of a creep in clayey soils is known since time immemorial. Creep of clayey soils started to be interesting for scientists and specialists after the observation of unacceptably large prolonged deformations, affecting normal exploitation of structures and roads.

It is possible to say that during the last century, and mainly during the last few years that creep deformations in clayey soils started to be one of the most important problems of soil mechanics. Today one can find lot examples of in-situ creep behaviour. A classical one is the uneven settlement of the Tower of Pisa in Italy. Due to the creep deformation of clays deposited in the form of lenses in the sandy base, the tower settled and tilted to one side. The mean settlement of the structure according to one of the many evaluations is 1.5 m, and the tower still continues to settle. An illustration of the tower's section with the geological situation below the structure together with the tower's mean settlement with mass of structure can be found in Figure 1.1 (a) and (b). Another classical civil engineering structure of 20th century is the Kansai International Airport of Japan. The Kansai Airport project has received considerable attention in the geotechnical engineering literature in part because of the scale of the project; Island I is 511 ha in approximately 18-m-deep seawater and Island II is 545 ha in approximately 20-m-deep seawater. The project is also significant because of the sufficiently detailed sub seabed information and observations of settlement and pore-water pressure reaching depths up to 350 m below the seabed.

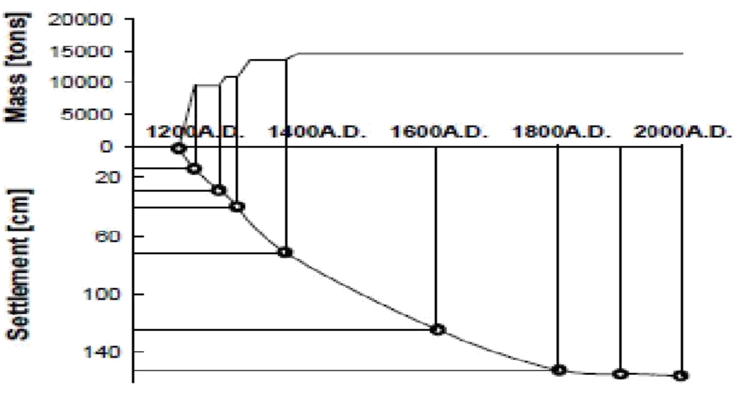
Hence, settlement is the important criterion which is to be considered in the design of any structure, the other being the strength of the soil deposit. The second component of total settlement (*i.e.* creep settlement) is neglected when the rate of settlement is low, but when the soil is highly plastic the secondary compression becomes predominant. The settlement analysis includes the determination of rate of settlement as well as the ultimate settlement. In the majority of cases, settlement analysis is restricted to only the primary consolidation, for which the results from laboratory consolidation tests on undisturbed samples are utilized. The rate of settlement is determined using the coefficient of consolidation, C_v, of the soil. In the literature it is mentioned that many factors cause secondary compression. Coefficient of secondary compression (C_α) is helpful in determining the secondary settlement with the assumption that compression varies linearly with log t in the secondary region. Generally it is seen that when the load is kept for a long time, the relationship between the creep strain (or void ratio) against log-time is linear and the slope of the linear portion of the curve is known as coefficient of "secondary" consolidation. But recently one of the new subjects arousing much controversy in the field of soil mechanics is the reliability of the linear relationship between the "secondary" compression and log time. The $log\ \delta$ - $log\ t$ method of representation consolidation data is useful when δ - $log\ t$ relationship in the secondary compression region is non- linear or it is not possible to differentiate between primary and secondary compression due to the masking effect of secondary compression (Sridharan and Prakash 1998).

This project is intended to investigate the nature and magnitude of the consolidation characteristics of representative prepared soil samples of varying plasticity. For the fundamental understanding of the compressibility of soil with a wide range of plasticity index, it is essential for developing theories of consolidation data for any foundation project. An assessment of the behaviour of coefficient of secondary consolidation (C_α) is urgently wanted.

a) Section of Tower of Pisa together with geological situation below the structure.



b) Illustration of the tower's mean settlement with mass of the structure.



Chapter-2

LITERATURE REVIEW

Chapter 2

Literature review

2.1 Introduction

The aim of this chapter is to encompass and synthesize the arguments and ideas presented by previous researchers on the compression behavior of soft soil. To understand these engineering properties of soil, knowledge of the major factors and parameters affecting it are required. Accordingly, this literature review will have a momentous spotlight on soft clays.

Hanna (1950) carried out research at Fouad I University, Cairo. The observed and theoretical settlements, based on the results of the consolidation tests were relatively quite close; but numerically the theoretical settlements in many buildings were 3–4 times the observed values. Research made seemed to indicate that the disturbance of swelling clays on removal from samplers accounted for an important part of the difference. This compelled the author for the development of a sampler that permitted the testing of the sample without removal from the sampler.

Olson and Mesri (1970) revealed that the mechanical and physiochemical effect controls the compressibility of bentonite. Surface friction, strength and flexibility of clay minerals are the mechanical effect that controls the compressibility of bentonite. On the other hand, the interaction between the particles and pore fluid through the diffuse double layer controls the physicochemical effect of the compressibility of bentonite. On application of vertical consolidation pressure to the mixture, it gets compressed due to the compression of the swollen bentonite. Bending, sliding, rolling and crushing of the soil also results in compression which in turn results in a higher value of primary compression index (C_c).

Sheeran and Krizek (1971) explained laboratory techniques and procedures which are to be followed with a slurry consolidometer apparatus to prepare a high quality homogeneous soil sample of known stress history. Specimens are trimmed from such samples which can be used to investigate various engineering properties of cohesive soil. Procedures to prepare slurries at Northwestern University are explained and the characteristics of a large series of consolidated blocks are described.

Mesri (1973) investigated considerable importance of secondary or delayed compression and noted that coefficient of secondary compression is the powerful tool to explain the secondary compression. Mineral composition and physicochemical environment have significant influences on the co-efficient of secondary compression. The difference between the coefficient of secondary compression in field and laboratory is K_0 condition in field. He discussed various factors such as mechanism of secondary compression, consolidation pressure, precompression, remolding, rate of increase in effective stress, temperature influencing the coefficient of secondary compression. He classified the soil based on Secondary compressibility.

Table 2.1 Classification of soils based on secondary compressibility

Coefficient of secondary compression C_α, as a percentage	Secondary compressibility
< 0.2	Very low
0.4	Low
0.8	Medium
1.6	High
3.2	Very low
> 6.4	Extremely high

Mesri and Godlewski (1977) analyzed consolidation data on samples of three natural soil deposits and found out for any natural soil a unique relationship exists between secondary compression index $C_\alpha = \Delta e/\Delta log\ t$ and the primary compression index $C_c = \Delta e/\Delta log\ \sigma$. For variety of natural soils the values of C_α / C_c are in the range of 0.025 -

0.10. The relation between C_α and C_c holds true at any time, effective stress and void ratio. C_α is strongly dependent on the final effective stress and is not a function of load increment ratio.

Sridharan and Prakash (1977) recommended *log* δ - *log t* method for determining the coefficient of the coefficient of consolidation (C_v). The proposed method is based on the observation of *log U- log T plot*. *Log* δ - *log t* method does not require the initial compression for obtaining C_v. The suggested method helps in identification of straight lines more clearly, the intersection of these lines is clear enough to obtain $t_{88.3}$. This method ignores the initial compression determination for calculating C_v.

Sridharan and Jayadeva (1982) showed that the compressibility of pure clays under external load not only depends on the negative charges and crystallite structure of clay minerals but also on the ion concentration, cation valency, dielectric constant and temperature of the pore fluid. When the concentration of cation in the pore fluid is low, the *log e–log p* relationship is linear, and the compression index is a direct function of void ratio and independent of clay type.

Sridharan and Rao (1982) reported that the secondary compression coefficient decreases with increase in effective stress (or strength), with decrease in pressure increment and with decrease in void ratio. Seven organic fluids, water and air of different dielectric properties were used to vary the inter particle forces in one dimensional consolidation tests. Tests were also conducted in which load increments, load increment ratio, and void ratio are varied.

Mesri and Choi (1985) carried out highly realistic analysis using concepts of traditional soil mechanics. One dimensional consolidation test on soft soil was conducted using standard odometer. Two fundamental assumptions i.e. uniqueness of end of primary consolidation, void ratio, effective stress relationship and validity of Darcy's law are considered for the one-dimensional concept. The compressibility parameters, preconsolidation pressure and coefficient of permeability are determined from odometer test with direct permeability measurement. Computation work using ILLICON gave a good agreement with field observation.

Mesri and Castro (1987) focused on C_α / C_c concept to describe the secondary compression behavior of soil. The value of C_α at each consolidation pressure is obtained from linear segment of *e - log t* plot immediately beyond the transition from primary to secondary compression. The C_c value is obtained corresponding to the same pressure range from the slope of end of primary (EOP) *e- log p* curve. C_α / C_c concept was applied to obtain equation for K_0, which predicts that there is increase in K_0 during secondary compression. $K_0 = 1$ cannot be reached during the geologic age of soft clay.

Barbour and Fredlund (1989) stressed on smectite behaviour of clay, which is significantly controlled by pore liquid composition. The interactions of clay soil with pore fluid cause changes in volume and shear strength by ion diffusion under constant external stresses. The compressibility decreases with increasing pore aqueous solution concentration.

Tan et al. (1990) Conducted several tests to study the surface settlement and density. They compared test results with predicted values. The sedimentation of clayey slurry is studied using Kynch's theory. The permeability-versus-void-ratio relationship of the slurry can be determined from its surface settlement versus time curve. The settling velocity is not only a function of concentration but also depends on the initial condition; an indication of the effect of flocculation.

Dhowian (1991) stressed on compressibility characteristics of Sabkha soils. It was found that the material exhibits a significant amount of secondary compression upon loading. Rheological model was utilized to describe secondary compression of Sabkha formation.

The consolidation test results and direct field measurement were compared with the primary and secondary compression predicted by Lo rheological model.

Fox et al. (1992) revealed long-duration odometer tests on Middleton peat, which showed the important contribution of creep to total settlement. Primary consolidation is rapid and creep compression accounts for the majority of settlement**.** As a result, compression curves have an unusual shape, and the end of primary consolidation is best determined by pore pressure measurement rather than graphical construction. Tertiary compression is clearly seen following secondary compression and indicates that C_α is not constant but increases in time under constant effective stress. A well-defined linear relationship between C_α and C_c has not been observed.

Abu-Hejleh and Znidarcic (1995) proposed desiccation theory for soft cohesive soils. They conducted experiments on soft china clay for overall analysis of soft fine grained soil after deposition. The theory provides information on the benefits of the self-weight consolidation and desiccation for varying environmental evaporation conditions and slurry deposition rates simultaneously. They formulated a finite strain equation for overall consolidation and desiccation process. The numerical analysis provides the rate of vertical and lateral deformations for soil layers undergoing consolidation and desiccation.

Leroueil (1996) presented fundamental and practical aspects of compressibility of clay. There are several factors affecting the compressibility of natural clays: strain rate, temperature, sampling disturbance, stress path, and structuring phenomena. The combined effect of strain rate and temperature has been quantified and a viscous strain $\Delta\mathcal{E}^*_s$ is proposed. The 24 h odometer tests were conducted to estimate the strain at the end of primary consolidation. In low compressibility clay $\Delta\mathcal{E}^*_s$ is small and hence neglected whereas for high compressibility clay $\Delta\mathcal{E}^*_s$ is considered.

Mesri et al. (1997) conducted a series of odometer tests on undisturbed specimen of Middleton peat with or without surcharging to study the secondary compression behaviour. The behaviour of this soil obeyed the C_α / C_c concept of compressibility. The value of C_α / C_c for Middleton peat was 0.052. The lowest values of C_α were encountered in the recompression range where Cc is small, and the highest values of C_α were observed at effective vertical stresses just past the preconsolidation pressure where C_c maximizes.

Al - Shamrani (1998) conducted series of one-dimensional consolidation test on natural and preloaded undisturbed samples taken from Sabkha soil. The test results shows that the secondary compression index (C_α) remains constant. The ratio of C_α to primary compression index (C_c) is within the range as published in literature i.e. 0.02 - 0.1. The C_α is strongly dependent on effective stress, initially the C_α increases rapidly as effective stress increases then remains constant or increases slightly.

Sridharan and Prakash (1998) proposed estimation of secondary settlement based on secondary compression factor ($m = \Delta loge/\Delta log\ t$) is more realistic for soil which exhibit non-linear secondary compression behaviour. In such cases $log\ \delta$ - $log\ t$ method of representation of consolidation data is beneficial in exhibiting linear secondary compression behaviour over an appreciably larger time span. The δ - $log\ t$ method is applicable for soils showing linear variation in secondary compression where C_α can be used as a reliable parameter lest the settlement will be underestimate. No relation was found between C_α and C_c.

Mohamedelhassan and Shang (2002) developed a vacuum and surcharge combined one dimensional consolidation model and concluded that the soil consolidation characteristics from the conventional one dimensional consolidation test can be used in the design of vacuum preloading systems, provided that the one-dimensional loading condition prevails. Terzarghi's one dimensional consolidation model was revisited. They used two natural clay soils from Ontario, Canada to study the vacuum and vacuum–surcharge consolidation behaviour. The one-dimensional model of vacuum and vacuum–surcharge consolidation describes the consolidation behaviour of the soils well. This can be used in the region where one dimensional consolidation exists.

Petry et al. (2002) reviewed some of the advances developed over the past 60 years in improving our understanding of the nature and methods of modifying and stabilizing expansive clay soils. They highlighted the historical development in clay soils and their effective stabilization for light structures and pavements. Further they reviewed the current state of the practice in clay stabilization for light structures and pavements and identified future research needs in these areas.

Mesri (2003) focused on the primary and secondary compression of saturated soil. One dimensional consolidation is subjected to incremental load. Primary compression takes place during increase in effective stress and secondary compression occurs at a constant effective stress. $(\partial e/\partial \sigma'_v)_t$ and $(\partial e/\partial t)\sigma'_v$ are not constant they are interrelated and both depend upon total rate of compression influenced by boundary condition.

Muntohar (2003) carried out laboratory one- dimensional consolidation tests and stressed on the swelling and compressibility characteristics of soil – bentonite mixtures. The swelling and compressibility characteristics generally increase with increase in bentonite percentage in bentonite, kaolinite and sand mixtures. The swelling behaviour occurs in two phases, primary and secondary. The swell behaviour follows a hyperbolic model curve during these stages of swelling. The swelling and compressibility characteristics are also affected by amount and size of non- swelling fraction.

Mesri and Vardhanabhuti (2005) conducted a large volume of reliable measurements of one-dimensional settlement. They observed in the laboratory and in the field for a wide variety of natural soil deposits. They determined that secondary compression index C_α =

$\Delta e/\Delta \; log \; t$ (therefore, also $\Delta S/\Delta \; log \; t$) may remain constant, decrease, or increase with time.

Phanikumar and Sharma (2007) studied the effect of fly ash on the volume change of two different types of clay, one a highly plastic expansive clay and the other a non expansive clay, also of high plasticity. Addition of fly ash to expansive soils reduced their swelling. Free swell index decreased on addition of fly ash. Swell potential and swelling pressure also decreased significantly with decreasing fly ash content. For the type of fly ash and expansive clays used, 20% fly ash content reduced FSI, swell potential and swelling pressure by about 50%.The compression index of both the expansive and non expansive clays decreased by about 50% with addition of 20% fly ash content, which implies addition of fly ash reduced compressibility characteristics of both expansive and non expansive clays. The time required for the end of primary consolidation and the beginning of secondary consolidation was shortened in clays blended with fly ash. The amount of settlement of structures built on fly ash-amended expansive clays decreases and the rate of settlement increases reducing the time required for reaching the final settlement.

Mohamad (2008) gave an advisory note on surcharge preloading, common mistakes while applying the surcharge load, fundamental aspects of consolidation. There are some pitfalls while applying the method such as inadequate site investigation and laboratory testing, inadequate surcharge or no removal of surcharge, inadequate stability during construction, designing for compensation of primary settlement only, and removing surcharge too early.

Tripathy et al. (2010) showed that a vertical pressure increase was more effective in reducing the water content and the void ratio for the bentonite studied. Mineralogy and the physico-chemical interactions between the clay particles and the pore fluid have a significant influence on the volume change behavior of clays due to an increase in vertical pressure.

Jain and Nanda (2010) reviewed the classical theories of secondary compression on clays. They explained why there is difference in values or sometimes match with settlement values of odometer tests. The load increment ratio in field is quite small as compared to laboratory test results and rarely reaches unity. The in-situ field stress is quite high compared to applied pressure increment. The main reason of increase in secondary compression in field is *H/2B*. If *H/2B* = 0.4 then its value matches with laboratory test results, for *H/2B* > 0.4 a correction factor α is multiplied to *H/2B.*

Bhattacharya and Basack (2011) suggested Installation of prefabricated vertical drains, followed by preloading that accelerates the consolidation of soft soil having low hydraulic conductivity and low shear strength thereby reducing the time period.

Hanna et al. (2011) proposed an analytical solution to one-dimensional consolidation caused by constant rate of loading, which is helpful in estimating the degree of consolidation and consolidation settlement more realistically. Terzarghi's solutions were developed for instantaneous loading applied at $t_0 / 2$, where t_0 is the duration of loading. This assumption does not give reliable estimates of the consolidation settlements. Terzarghi's one dimensional theory is extended to constant rate of loading. Expression relating average degree of consolidation and time factor is derived.

Widodo and Ibrahim (2012) conducted laboratory test on Pontianak soil and reported that the value of primary compression index (C_c) is 0.755, and is in the range from 0.279 to 1.663 which indicates soft clay. C_c value when calculated using water content and liquid limit (i.e. empirical equation) gave an underestimated value when compared with laboratory test result. Calculation of C_c from void ratio gave better result. The primary compression index for several kinds of soil is tabulated below.

Table 2.2 The primary compression index for several kinds of soil

Kind of soil	C_c
Dense Sand	0.0005 – 0.01
Loose Sand	0.025 – 0.05
Firm Clay	0.03 – 0.06
Stiff Clay	0.06 – 0.15
Medium – Soft Clay	0.15 – 1.0
Organic Soil	1.0 – 4.5
Rock	0

Pal and Ghosh (2014) presented the consolidation and swelling characteristics of fly ash–montmorillonite clay mixes. Nine types of fly ash samples collected from different thermal power plants of the Eastern part of India have been used in this study. The amount of Montmorillonite clay added to each fly ash sample is 10, 20, 30, 40, and 50%. The compression index (C_c) of the fly ashes indicates that the rate of consolidation is very fast. With an increase in montmorillonite clay content from 0 to 50%, the compressibility of the fly ash–montmorillonite clay mix increases, irrespective of the type of fly ash. The value of the compression index (Cc) of montmorillonite clay indicates that the embankments and fills made of fly ash–montmorillonite clay mixtures (i.e., 30, 40, and 50% of montmorillonite clay in the mix) and the structures constructed on such fills would suffer large deformation, whereas fly ash and fly ash mixed with 20% montmorillonite clay would not suffer large deformation.

2.2 Summarized critical review

The literatures as presented in section 2.1 reveal that the majority of previous research in this area has focused on the behavior of natural clays. The Primary compression index (C_c) and secondary compression (C_α) is influenced by many factors such as mineral composition, physicochemical environment and pore fluid. The C_α is strongly dependent on effective stress and may remain constant, decrease, or increase with time. Installation of prefabricated vertical drains, followed by preloading that accelerates the consolidation of soft soil. For any natural soil a unique relationship exists between secondary compression index $C_\alpha = \Delta e/\Delta \log t$ and the primary compression index $C_c = \Delta e/\Delta \log \sigma$. Tables 2.1 and 2.2 show classification of soils based on secondary compressibility and the primary compression index for several kinds of soil respectively. For variety of natural soils the values of C_α / C_c are in the range of 0.025 - 0.10. The difference between the coefficient of secondary compression in field and laboratory is K_0 condition in field. C_α / C_c concept was applied to obtain equation for K_0, which predicts that there is increase in K_0 during secondary compression.

2.3 Factors affecting compressibility of soft soil

- The soil type and its structure
- The effective stress
- The Influence of pore fluid

2.4 The soil type and its structure

A granular soil compresses immediately upon loading but the compression is relatively small. Because of its high permeability it does not take much time for pore water to drain out whereas fine grained soils exhibit time dependent consolidation and the compression is quite high.

2.5 The effective stress

The fine grained soils have a relatively smaller bearing capacity than coarse grained soils. So they have a greater degree of compressibility. Highly plastic soils exhibit significant volume change on application of vertical stress. There will be squeezing out of pore water causing a time dependent decrease in volume. Odometer tests are conducted to establish relationship between effective stress and void ratio. When the concentration of cation in the pore fluid is low, the log *e*–log *p* relationship is linear, and the compression index is a direct function of void ratio and independent of clay type. The void ratio decreases with increase in vertical stress.

2.6 The Influence of pore fluid

The compressibility of clay is greatly influenced by pore fluid (Sridharan and Jayadeva 1982). Highly plastic soil generally comes in contact of water and swell. There is a chance the water may come in contact of different cation, organic fluids and aqueous solutions. The Compressibility generally decreases with increasing pore aqueous solution concentration (Barbour and Fredlund 1989).

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2.7 Scope and Objective of the present study

A review of past studies has revealed that the majority of previous research in this area has focused on the behavior of natural clays, while research on soft clays with wide range plasticity index has been limited. Previous researchers basically focused on compressibility behaviour of natural clayey soils of different regions. Hence the study was limited to that region. The scope of the present study is:

- The importance of secondary consolidation is predominant for highly plastic soils.
- The Prediction of secondary consolidation is difficult as compared to primary consolidation.
- The correlation of primary compression index (C_c) with different factors like plasticity index (I_p) is limited.
- The C_α / C_c law of compressibility is valid for natural soils.
- No definite correlation exists between coefficient of secondary consolidation and other parameters.
- Besides C_α and C_c, the importance of other parameters like m_v, C_v, k on behaviour of soft clay and C_α / ($1+e_p$) for varying I_p have not been given due importance.

The fundamental understanding of the compressibility of soil with a wide range of plasticity index is essential for developing theories of consolidation data for any foundation project. An assessment of the behaviour of C_α with time is urgently wanted. Keeping these in mind a series of odometer tests, with utmost care is conducted in laboratory by mixing different percentages of silt with commercially available Na and Ca bentonite.

The main objectives are:

- To determine the primary compression index (C_c) and secondary compression index (C_α) of soils with wide margin of plasticity index (I_p).
- The study the variation of C_c and C_α with plasticity index and stress range.
- To check the applicability of C_α / C_c law of compressibility for soils with wide range of plasticity index (I_p) i.e. for low to high plastic soils.
- To observe the dependency of consolidation parameters like C_v, m_v, k with factors such as plasticity index (I_p) and consolidation stress range.
- Establish correlation of C_c with I_p and C_α with I_p and consolidation pressure if any.
- Establish correlation between C_α/ ($1+e_p$) with I_p if any.

CHAPTER 3

MATERIAL AND METHODS

Material and methods

3.1 Material

This chapter describes the methodology and materials used to achieve the objectives. Soil samples have been prepared by mixing commercially available highly plastic soils (Na and Ca bentonite) with silt in different proportions. Commercially available Na and Ca bentonite were collected from Rourkela, Odisha and Bikaner, Rajasthan respectively. Silty soil was collected from Koel river site, Rourkela, Odisha. Procedure for sample preparation, sampling and testing techniques used for characterization of materials as well as experimental setup for investigation are reported in the following section.

3.2 Methods

Grain size distribution, specific gravity of the samples were determined according to IS: 2720- part 4 (1975), IS: 2720-Part 3 (1980) respectively. The consistency limits were determined as per IS: 2720-Part 5 (1985). The test results were repeated thrice and the average values were presented.

3.2.1 Grain Size Distribution

The grain size distributions of Na, Ca bentonite and silt were determined in accordance with IS: 2720- part 4 (1975). The grain size distribution curve of Na and Ca bentonite are shown in Fig. 3.1and 3.2 respectively. The grain size distribution curve of silt is shown in Fig. 3.3.

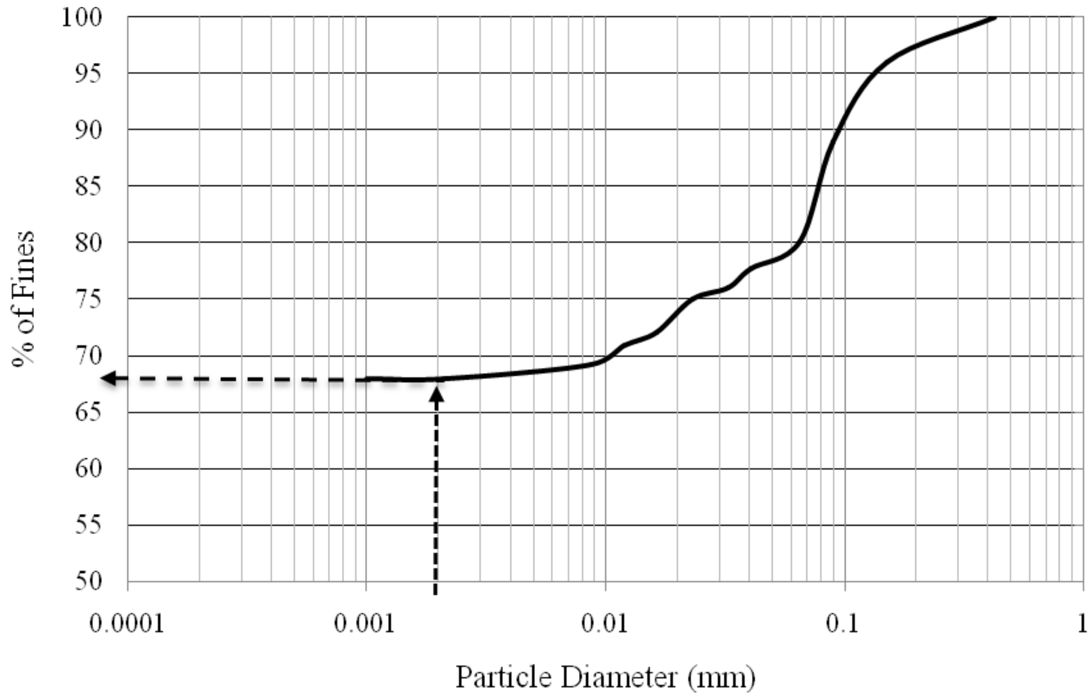

Figure 3.1 Grain size distribution of Na bentonite

From the above graph the percentage of clay in Na bentonite was found to be 68%.

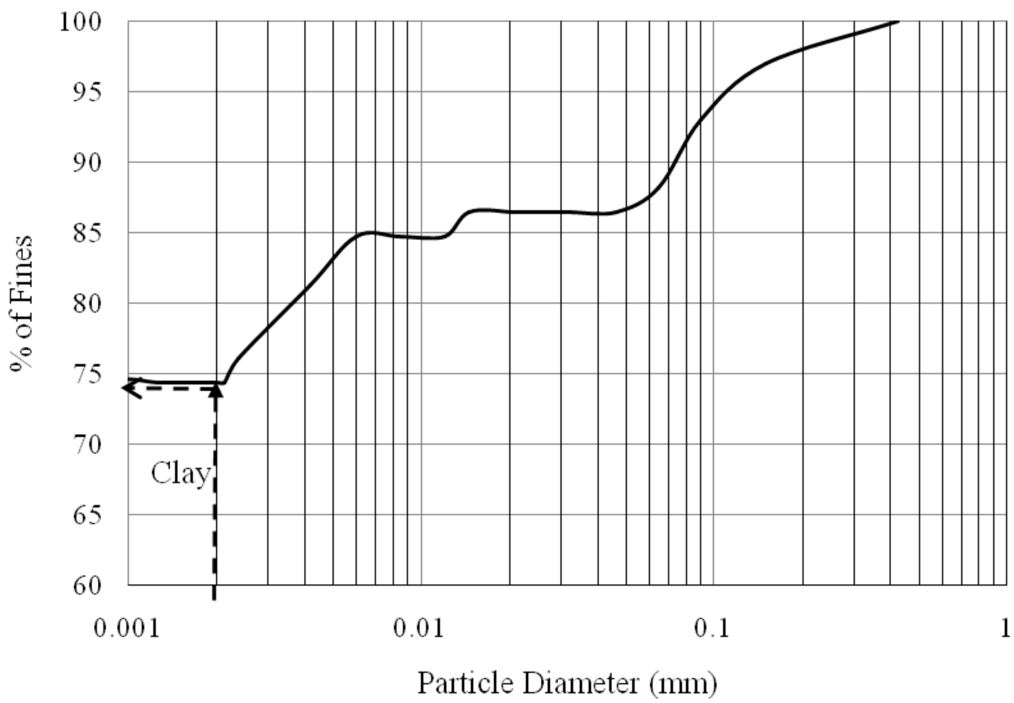

Figure 3.2 Grain size distribution of Ca bentonite

From the graph in Figure 3.2 the percentage of clay in Ca bentonite was found to be 73%.

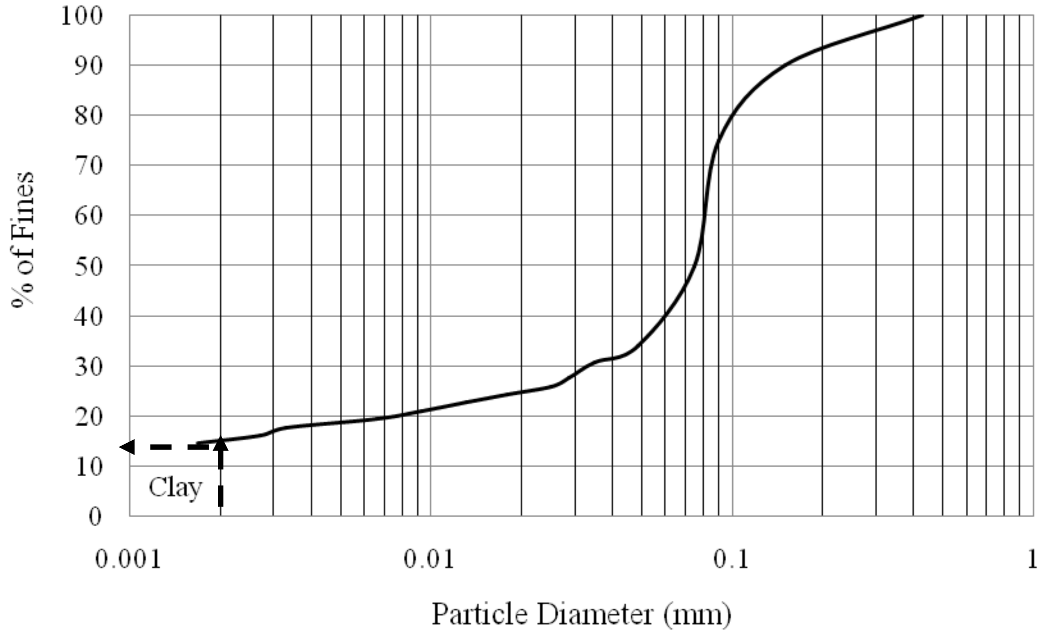

Figure 3.3 Grain size distribution of silty soil

From the graph in Figure 3.3, the percentage of sand, silt and clay was found to be 50%, 35% and 15% respectively.

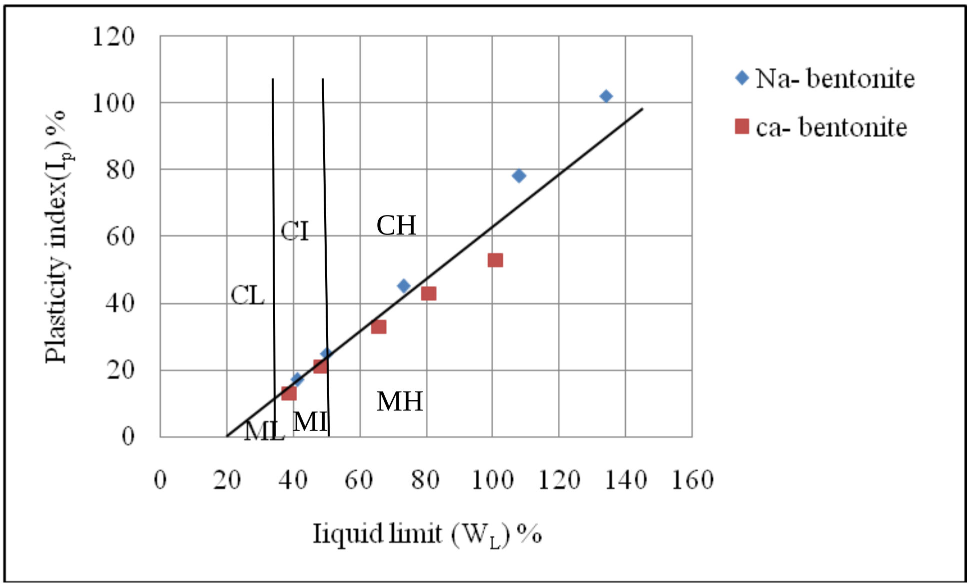

Figure 3.4 Classification of Na-bentonite and Ca-bentonite in the plasticity chart

From the chart in Figure 3.4, Na- bentonite silt mixes are classified as CH-CI and Ca-bentonite silt mixes are classified as MH-MI.

3.2.2 Specific Gravity

The specific gravity of Na bentonite and Ca bentonite, silt were determined as per IS: 2720-Part 3 (1980) and their values were found to be **2.7**, **2.67** and **2.6** respectively.

3.2.3 Atterberg Limits

Atterberg limits were determined as per IS: 2720-Part 5 (1985) and the results are tabulated in Table 3.1.

Table 3.1 Atterberg limits of soil

Soil	Liquid limit (w_l %)	Plastic limit (w_p %)	Plasticity Index (I_p %)
Na- Bentonite	407	42	365
Ca- Bentonite	140	61	79
Silt	27	22	5

Table 3.2 consistency limits of Na and Ca bentonite soil mixed with silt in different proportions

Na- Bentonite				
Soil	Proportion (bentonite: silt)	Liquid limit (w_l%)	Plastic limit (w_p%)	Plasticity Index (I_p%)
A	50:50	134	32	102
B	40:60	108	30.	78
C	30:70	73	28	45
D	20:80	50	25	25
E	10:90	41	24	17
Ca- Bentonite				
F	50:50	101	48	53
G	40:60	81	38	43
H	30:70	66	33	33
I	20:80	48	27	21
J	10:90	39	26	13

3.2.4 XRD ANALYSIS

The mineral composition of bentonite was determined by X-ray diffraction method. According to Bragg's law, the XRD identifies the minerals based on the relationship between the angle of incidence of the X-rays (θ) to the c-axis spacing (d). A Philips automated powder diffractometer was used for XRD analysis in this study.

Fine grained bentonite power and silt of 1.5 g was kept in oven drying for 2 hours and allowed to cool in room temperature. Then, sample is filled in the sample holder of diffractometer and the XRD pattern is obtained by scanning over angles ranging from 5° to 90°, 2θ at 5°/min. In the step mode, a 0.05° - 2θ step for 2 s was given. Results are analyzed using X-pert High Score software. From the results of XRD analysis mineral composition such as Illite, Kaolinite, Montmorillonite, Muscovite are found in Na bentonite sample.

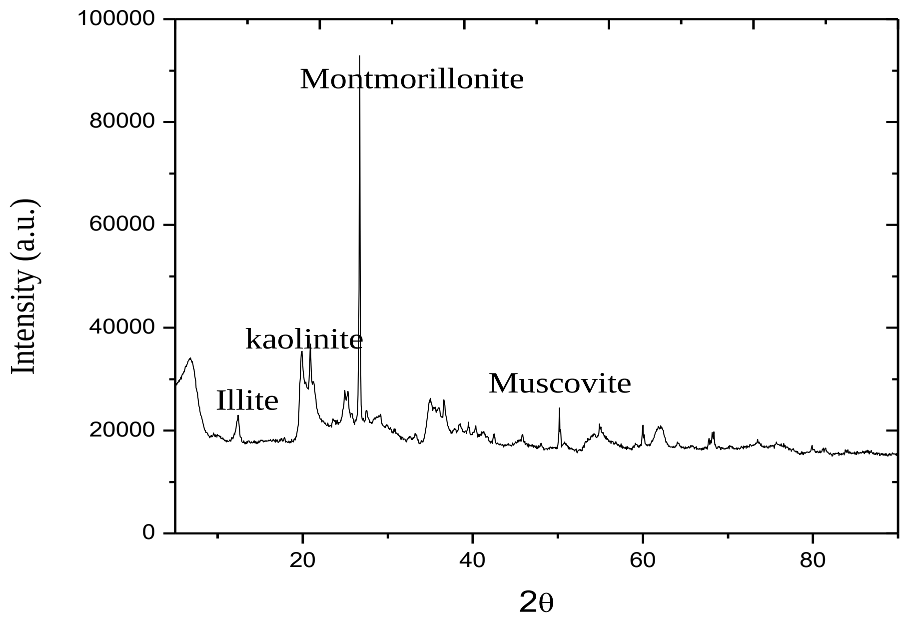

Figure 3.5 XRD analysis showing mineralogical composition of Na bentonite

Similarly, the analyses were carried out for Ca bentonite and silty soil and the mineral composition were determined.

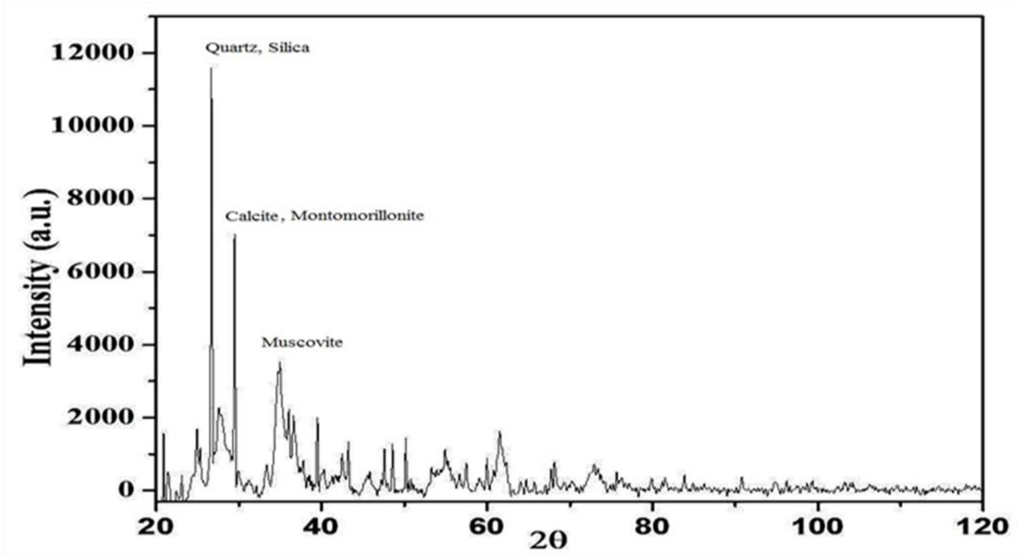

Figure 3.6 XRD analysis showing mineralogical composition of Ca bentonite

From the above analysis, in Figure 3.6 the compound present in Ca bentonite were Quartz, Montmorillonite, Muscovite and Calcite.

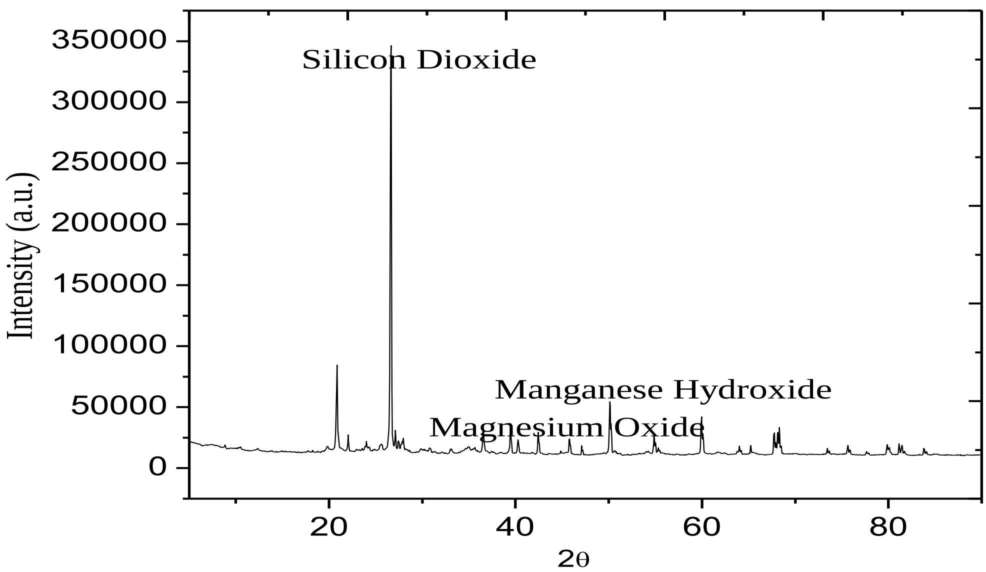

Figure 3.7 XRD analysis showing mineralogical composition of Silty soil

Silicon dioxide, Manganese hydroxide, Magnesium oxide were the mineralogical composition of silty soil showcased in the analyses above.

3.2.5 SEM AND EDX ANALYSIS

The individual morphology and particle chemistry of Na and Ca bentonite, silt were studied in SEM fitted with EDX. Scanning Electron Microscopy uses a focused beam of high-energy electrons to generate a variety of signals at the surface of solid specimens.

Fine grained bentonite powder and silt of 1.5 g were kept in oven drying for 2 hours and allowed to cool in room temperature. Then, sample was filled in the sample holder of SEM. Data was collected over a selected area of the surface of the sample and a two-dimensional image was generated that displayed spatial variations in properties including chemical characterization, texture and orientation of materials. The Morphology and orientation of Na, Ca-bentonite and silt are shown in Fig.3.8, Fig.3.9, and Fig.3.10 respectively.

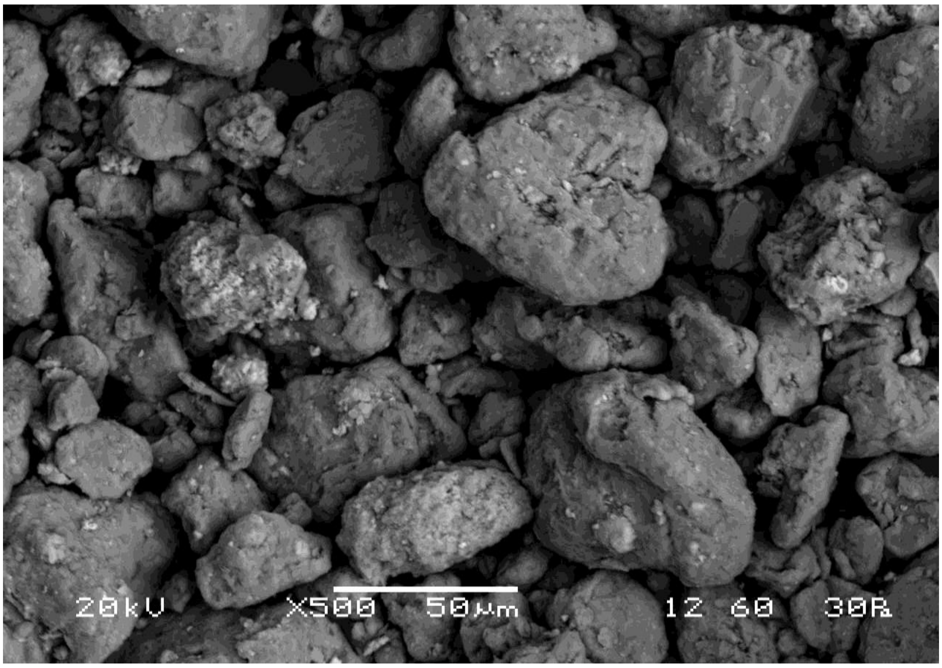

Figure 3.8 Morphology of Na bentonite

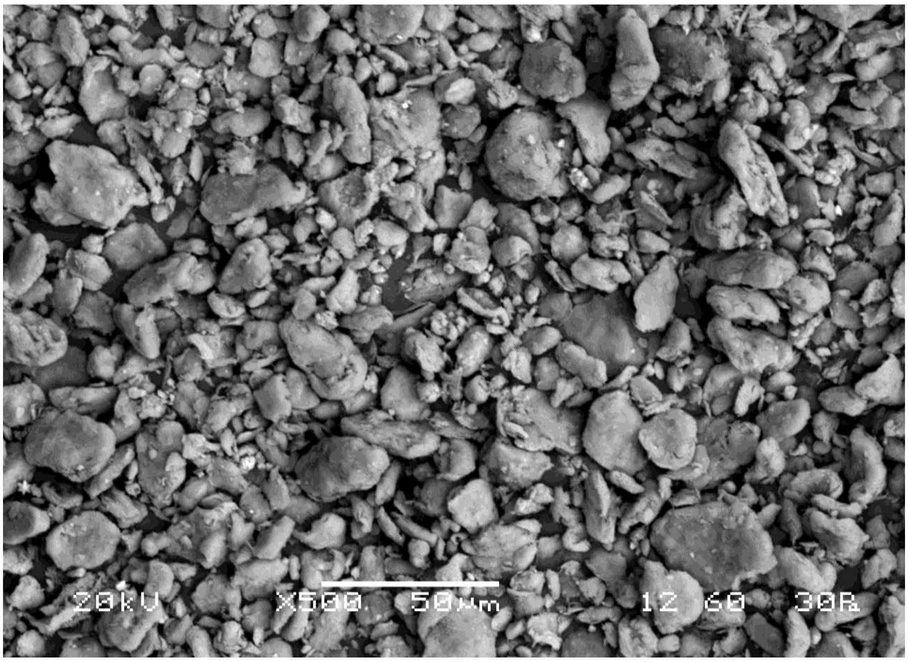

Figure 3.9 Morphology of Ca bentonite

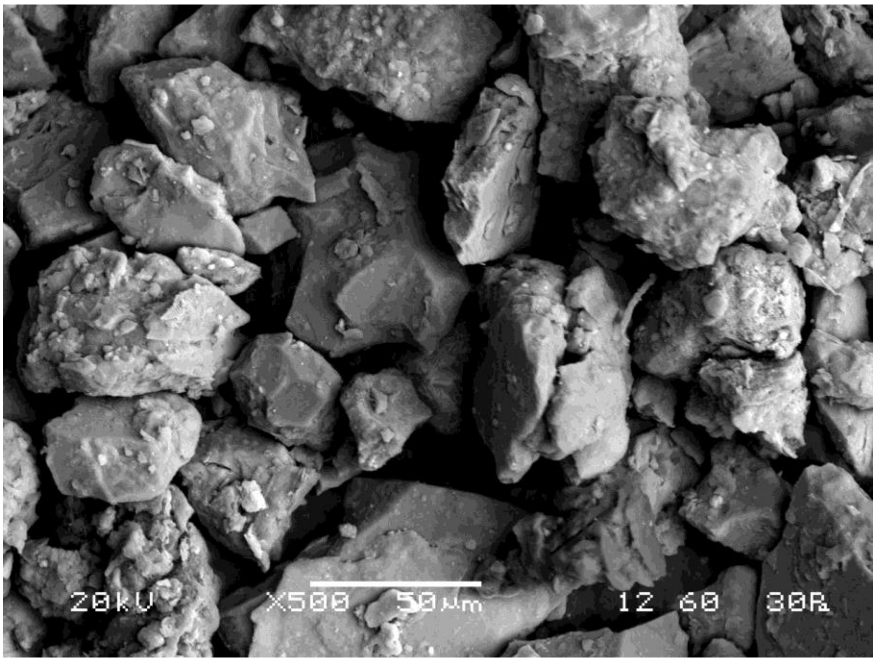

Figure 3.10 Morphology of silt

The EDX detector separates the characteristic X-rays of different elements into an energy spectrum and EDX system software is used to analyze the energy spectrum in order to determine the abundance of specific elements. A typical EDX spectrum is portrayed as a plot of X-ray counts vs. energy (in keV). Energy peaks correspond to the various elements in the sample.

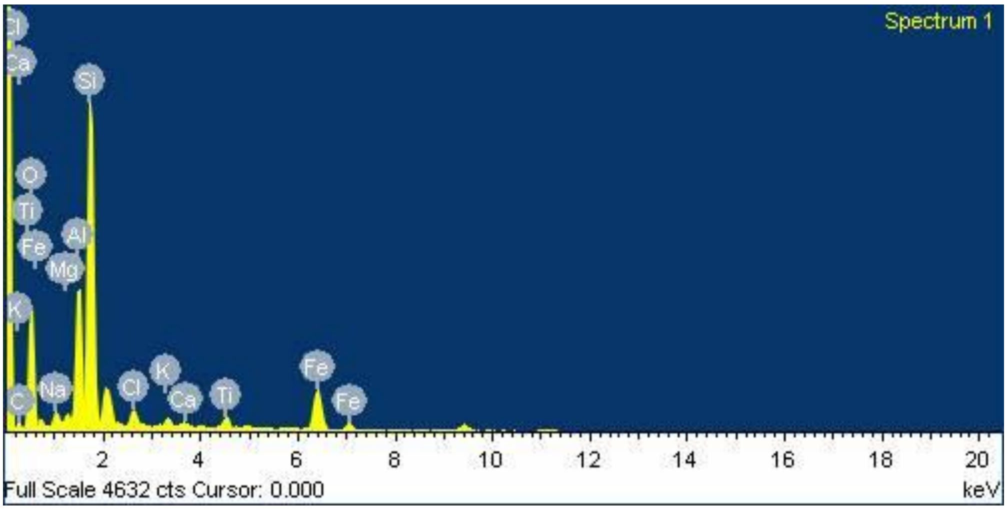

Figure 3.11 EDX spectrum of Na bentonite

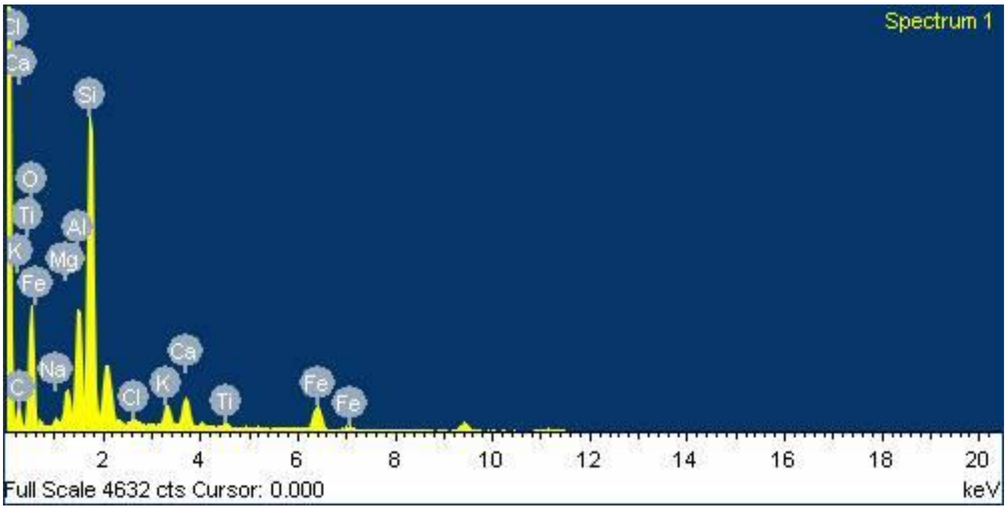

Figure 3.12 EDX spectrum of Ca bentonite

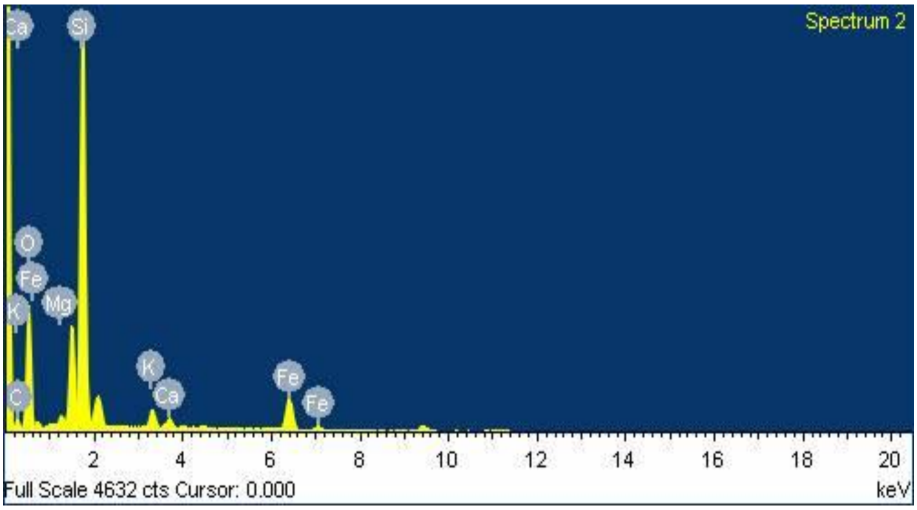

Figure 3.13 EDX spectrum of silt

3.3 One-Dimensional Laboratory Consolidation Test

The one-dimensional consolidation testing procedure was first suggested by Terzaghi (1925). The test is performed in a consolidometer (oedometer). Figure 3.14 shows one dimensional consolidometer. The soil specimen is placed inside a metal ring with two porous stones, one at top of the specimen and another at the bottom. The load is applied through a lever arm, and compression is measured through dial gauge. The specimen is kept under water during the test. Each load is usually kept for 24 hours. After that the load is usually doubled and the compression measurement is continued.

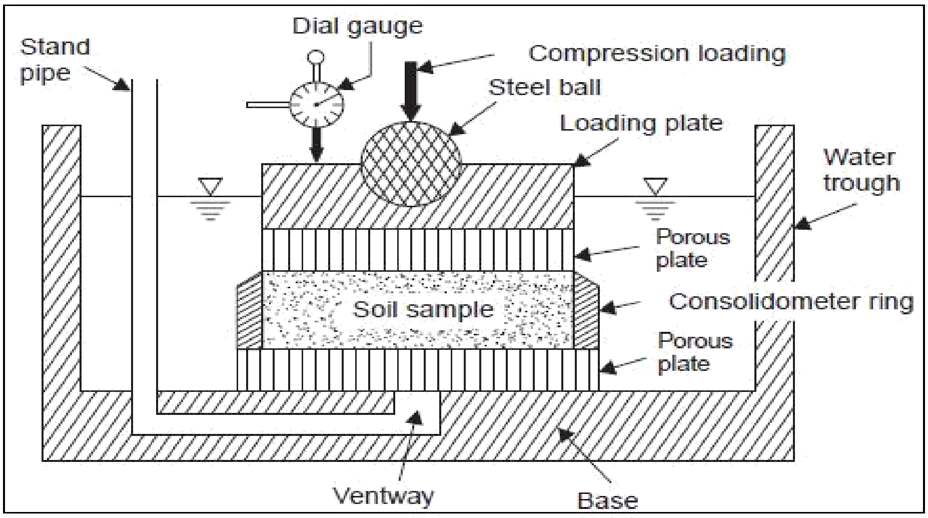

Figure 3.14 One Dimensional Consolidometer

3.3.1 Sequence of experiment

To prepare the representative sample for oedometer test, oven dried Na bentonite; Ca bentonite and silt were taken. The dry sample was then mixed homogeneously at different proportion (i.e. bentonite: silt) in a tray. Different proportions of bentonite and silt such as 50: 50, 40: 60, 30: 70, 20: 80, 10: 90 were taken. The liquid limit of individual sample was found out for each soil mixture. Thereafter, each bentonite and silt mixture was added with about 2 to 2.5 times the liquid limit of the corresponding soil to get uniform slurry.

In order to prepare the soil samples with different plasticity, PVC pipes of 150 mm diameter and 200 mm height were taken and placed over a tray. Porous stones were placed at the bottom of the PVC pipe followed by filter paper. Prepared soil slurry was poured into the PVC pipe over which filter paper with porous stone was placed at the top of the filled slurry in the PVC pipe. Water from the slurry was taken out gradually by leaving the complete setup under self weight. After three to four hours, a surcharge load of 2 kPa was provided to remove water from the soil slurry put inside the PVC pipe to form soft soil cake. The whole process of soil cake formation took about 6 - 7 days. Thereafter, the surcharge load was removed and the porous stone and filter paper removed from the soil cake formed inside the PVC pipe. The PVC pipe was the slowly taken out with utmost care keeping the soil cake on the tray. Thus a homogeneous soil cake was developed with different water content for different bentonite silt mix. A sample extractor of 75 mm diameter and 25 mm thickness was used to extract the sample from the soil cake so formed for each soil mix. The soil sample collected was placed in a fixed ring oedometer for testing. Filter papers with porous stones were placed at the top and bottom of the sample. The entire assembly was placed in the consolidation cell and positioned in the loading frame. Sample prepared with each proportion was repeated 3 times to check the accuracy.

The photos given in Figure 3.15 through 3.35 explain the sequence of preparing soil samples of different I_ps and the tests in the oedometer.

Figure 3.15 Dry samples of bentonite and silt

Figure 3.16 Mixing of dry sample

Figure 3.17 Addition of water

Figure 3.18 Mixing of wet sample

Figure 3.19 Preparation of slurry

Figure 3.20 Placing porous stone at the bottom of PVC pipe

Figure 3.21 Placing filter paper over the bottom porous stone

Figure 3.22 Pouring the slurry into the PVC pipe

Figure 3.23 Placing filter paper over the slurry

Figure 3.24 Placing porous stone over the slurry

Figure 3.25 Placing surcharge load over porous stone

Figure 3.26 Removal of surcharge load after 6 to 7 days

Figure 3.27 Removal of PVC pipe after 6 to 7 days

Figure 3.28 Removal of porous stone from top

Figure 3.29 Removal of filter paper from top

Figure 3.30 Insertion of sample extractor into the cake

Figure 3.31 Trimming out the edges of the cake

Figure 3.32 Extracted undisturbed sample

Figure 3.33 Transferring the extracted sample into Oedometer

Figure 3.34 Assembling the consolidation cell

Figure 3.35 Placing the sample in the loading frame

3.3.2 Test Procedure

Following the sequences as presented in 3.3.1, the one- dimensional consolidation test was conducted in accordance with ASTM D2435/ (ASTM, 1999). Standard fixed ring type consolidometer was used with ring diameter 75mm and 25mm height to perform the experiment. The soil specimen was kept inside the metal ring, with a porous stone at the top and bottom. All specimens were tested at moisture content of extracted soil sample. Utmost care was taken to prevent air entrapment in the soil specimens remolded in the rings. The specimen was kept under water throughout the test. The load on the specimen was applied through a lever arm, and the compression of the specimen was measured by a dial gauge. The deformation of the soil sample was noted from the dial gauge starting from 0 s, 60 s, 1 min, 1.5 min, 2 min, 4 min, 6 min, 8 min, 10 min, 15 min, 30 min, 60 min, 90 min, 120 min, 150 min, and 1440 min. The load was usually doubled every 24 h (i.e. incremental loading of 50, 100, 200, 400, 800, 1600 kPa were applied). At the end of the test, for each incremental loading, the stress on the specimen was the effective stress $\sigma^{\prime}$. Once the specific gravity of the soil solids, the initial specimen dimensions, and the specimen deformation at the end of each load was determined, the corresponding void ratio was calculated. For each load increment, the specimen deformation and the corresponding time t was plotted on semi logarithmic graph. The primary compression index was determined from the slope of e versus $\log \sigma'$ graph at the location where primary consolidation ends.

The Figures 4.1 and 4.2 show the e versus $\log \sigma'$ graph for the soil prepared with different plasticity index (I_p). The secondary compression index was determined from the linear portion of settlement (δ) - log t plot.

CHAPTER 4

RESULTS AND DISCUSSIONS

Chapter4

Results and Discussions

4.1 Introduction

Tests were conducted on ten numbers of plastic clays, prepared by mixing different percentages of silt with commercially available Na and Ca bentonite to have representative samples of different plasticity. One dimensional consolidation tests were carried out for each of these soil samples at different vertical effective stresses. The dial gauge readings corresponding to each incremental loading were noted down. Each incremental load was maintained for at least 24 hours until the deformation ceases. From the dial gauge reading, the void ratios corresponding to different stress were calculated. Then void ratio versus logarithm of vertical effective stress was presented for each soil. The primary compression index (C_c) was calculated from the slope of e versus log σ' curve. From the consolidation test data, the values of different parameters like primary compression index (C_c), secondary compression index (C_α), time rate of consolidation (C_v), coefficient of volume compressibility (m_v), permeability (k) were determined. The parameters were studied for factors like stress range, plasticity index and final void ratio. The primary compression index (C_c) for each soil was calculated from the slope of e versus log σ' plot. The secondary compression index was calculated for each soil at various stress range of 0-50 kPa, 50-100 kPa, 100-200 kPa, 200-400 kPa, 400-800 kPa, 800-1600 kPa from the plots of settlement versus log t plots. Various graphs are plotted such as log σ' versus final void ratio (e_f), settlement versus log t from observed data to determine the primary compression (C_c) and secondary compression index (C_α). Each parameter was examined vividly to figure out the trend with plasticity index (I_p %) and stress range.

4.2 Consolidation test with Na and Ca bentonite blended with silt

Consolidation tests were conducted with samples obtained by mixing Na and Ca bentonite with silt in different proportion and data obtained from tests were shown in Tables 4.1 and 4.2 respectively. From the tables it is observed that the void ratio decreased with increase in pressure and increased with increase in plasticity index. Initially the void ratio of the soil (Na-bentonite mixed with silt) corresponding to plasticity index (I_p %) 102, 78, 45, 25 and 17 were 4.24, 3.69, 2.97, 2.22 and 1.35 respectively. The final void ratios of the soil were 1.52, 1.95, 1.44, 1.25, and 0.91 respectively after the completion of consolidation tests. Similarly the initial void ratio of soil (Ca bentonite mixed with silt) corresponding to plasticity index (I_p %) of 53, 43, 33, 21, 13 are 2.78, 2.27 and 1.97. The final void ratios of the soil were 1.55, 1.17, 1.01, 0.74, and 0.7 respectively. The void ratio corresponding to logarithmic of consolidation pressure for both soil samples (Na and Ca bentonite mixed with silt) were plotted in Figures 4.1 and 4.2 respectively.

Table 4.1 Final void ratios of soil (Na-bentonite mixed with silt) corresponding to plasticity index and consolidation pressure

Plasticity index (I_p %)	Effective vertical stress (kPa)	Final void ratio (e_f)
102	0	4.24
	50	3.03
	100	2.59
	200	2.25
	400	1.97
	800	1.77
	1600	1.52
78	0	3.69
	50	2.89
	100	2.61
	200	2.4
	400	2.22
	800	2.07
	1600	1.95
45	0	2.97
	50	2.22
	100	2.05
	200	1.84
	400	1.68
	800	1.55
	1600	1.44

Table 4.1 (Continued)

Plasticity index (I_p %)	Effective vertical stress (kPa)	Final void ratio (e_f)
25	0	2.22
	50	1.75
	100	1.63
	200	1.52
	400	1.41
	800	1.33
	1600	1.25
17	0	1.35
	50	1.23
	100	1.18
	200	1.11
	400	1.04
	800	0.98
	1600	0.91

Table 4.2 Final void ratios of soil (Ca bentonite mixed with silt) corresponding to plasticity index and consolidation pressure

Plasticity index (I_p %)	Effective vertical stress (kPa)	Final void ratio (e_f)
53	0	2.78
	50	2.33
	100	2.0
	200	1.8
	400	1.7
	800	1.5
	1600	1.4
43	0	2.27
	50	1.81
	100	1.66
	200	1.51
	400	1.4
	800	1.27
	1600	1.17
33	0	1.97
	50	1.61
	100	1.47
	200	1.34
	400	1.21
	800	1.1
	1600	1.01

Table 4.2 (Continued)

Plasticity index (I_p %)	Effective vertical stress (kPa)	Final void ratio (e_f)
21	0	1.57
	50	1.3
	100	1.21
	200	1.11
	400	1.01
	800	0.93
	1600	0.74
13	0	1.22
	50	1.02
	100	0.96
	200	0.89
	400	0.83
	800	0.78
	1600	0.7

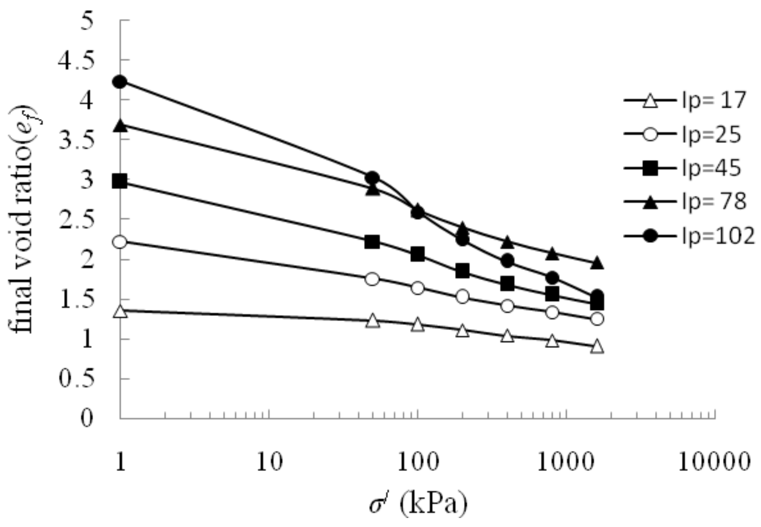

Figure 4.1 *e* - *log* σ^{l} curves for the consolidation test conducted for soil with Na bentonite mixed with silt

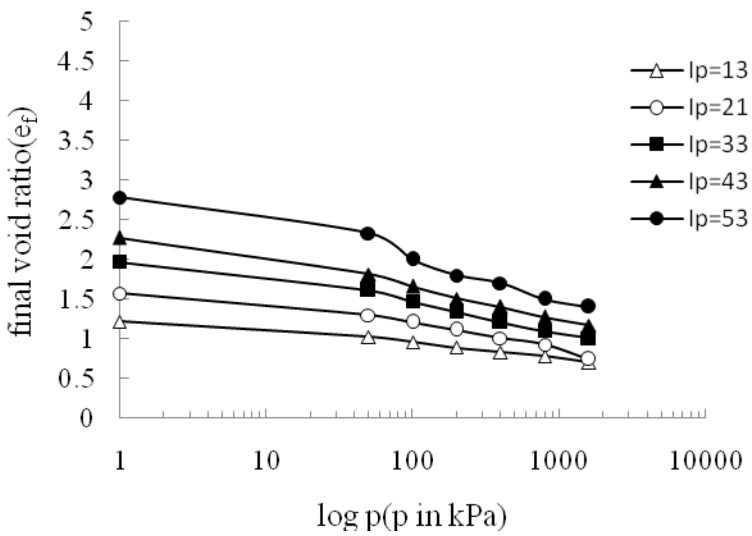

Figure 4.2 *e* - *log* σ^{l} curves for the consolidation test conducted for soil with Ca bentonite mixed with silt

The settlement corresponding to different time intervals at various consolidation stress range were plotted for each soil. These are shown in Figures 4.3 through 4.7 for Na bentonite mixed with silt and Figure 4.8 through 4.12 for Ca bentonite mixed with silt. From the settlement versus *log t* plots, it is observed that in general settlement increases with increase in loading. From the Figures 4.3 through 4.12 it is seen that the rate of increase of settlement is more at lower stress range as compared to higher stress range. From Table 4.3 as the plasticity of the soil increases, the settlement increases. This holds true for Na and Ca bentonite mixes.

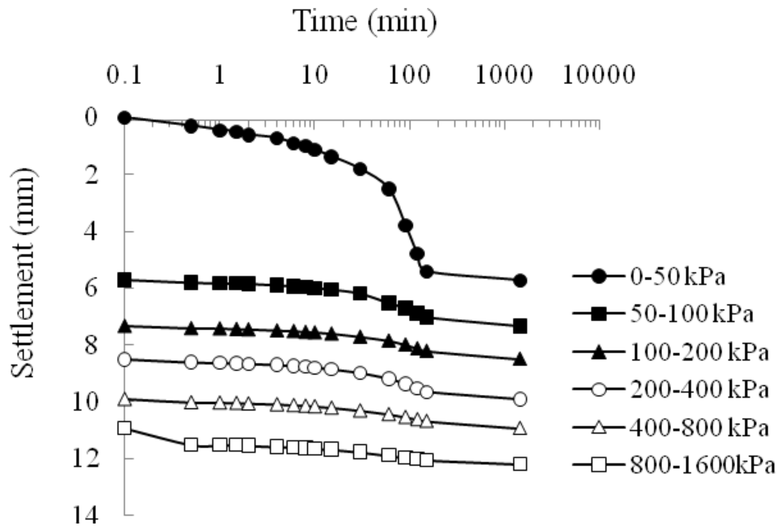

Figure 4.3 Settlement versus *log t* curve for sample with plasticity index 102

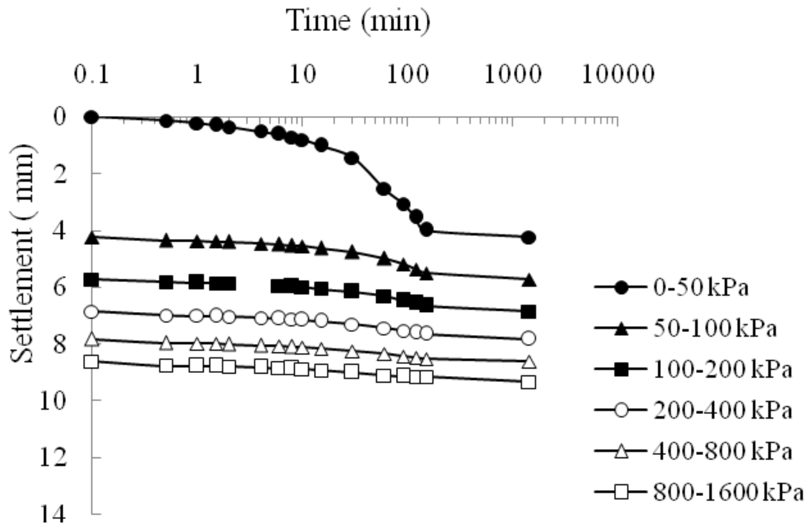

Figure 4.4 Settlement versus *log t* curve for sample with plasticity index 78

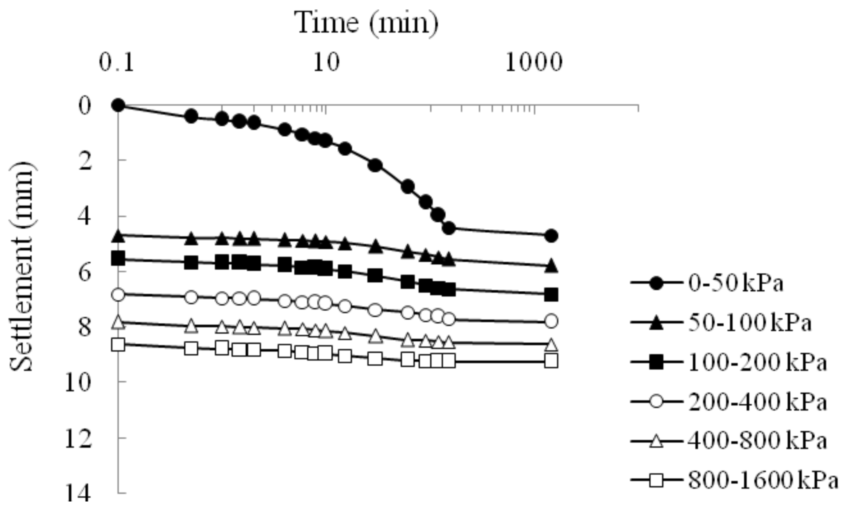

Figure 4.5 Settlement versus *log t* curve for sample with plasticity index 45

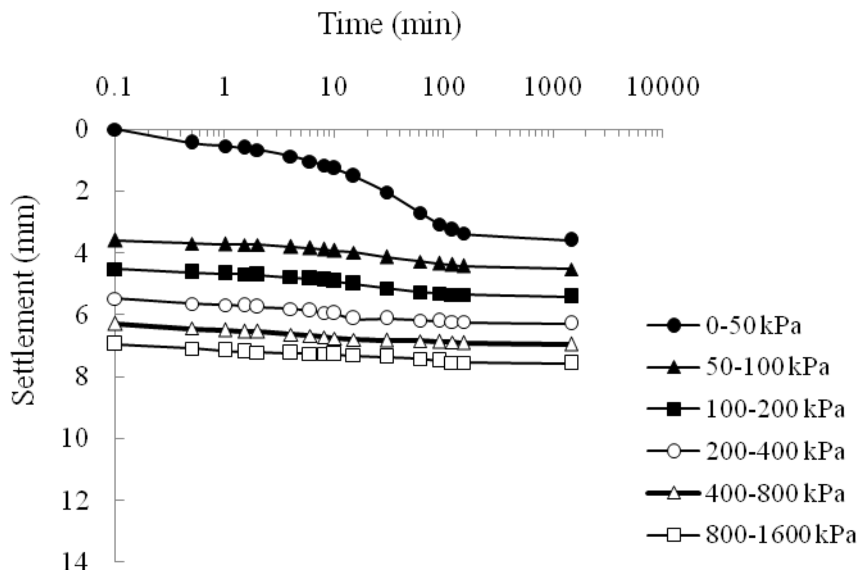

Figure 4.6 Settlement versus *log t* curve for sample with plasticity index 25

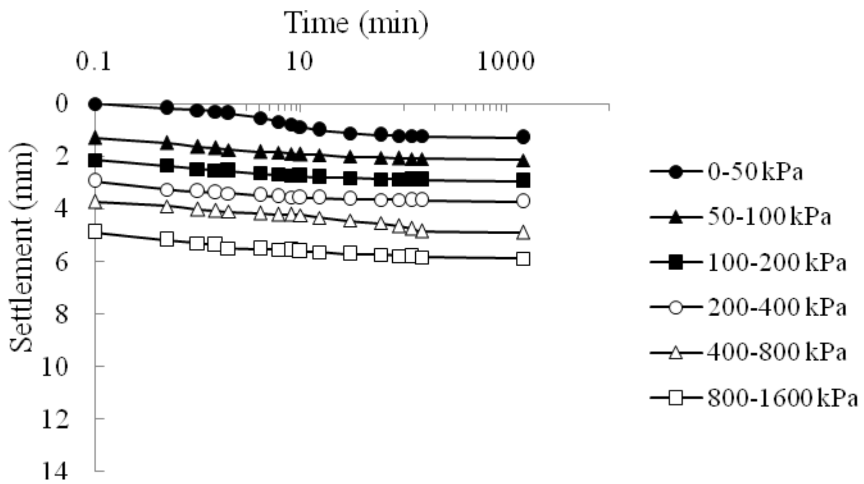

Figure 4.7 Settlement versus *log t* curve for sample with plasticity index 17

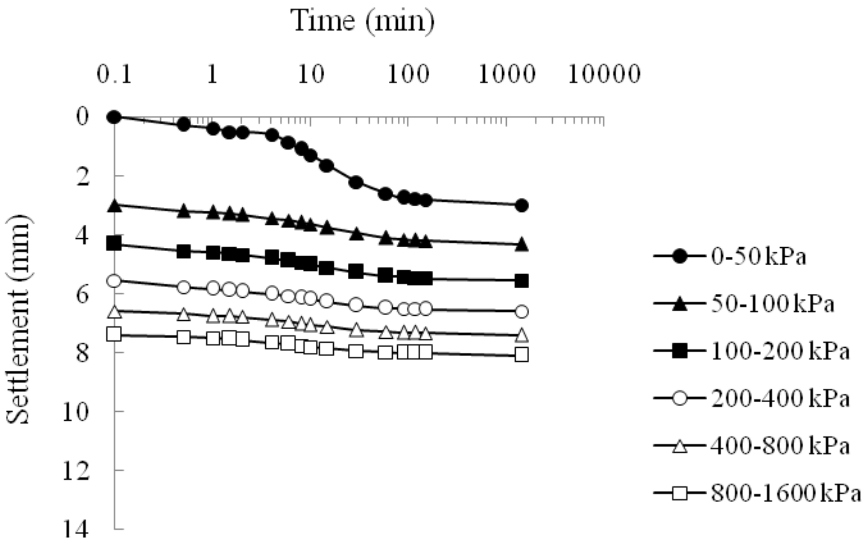

Figure 4.8 Settlement versus *log t* curve for sample with plasticity index 53

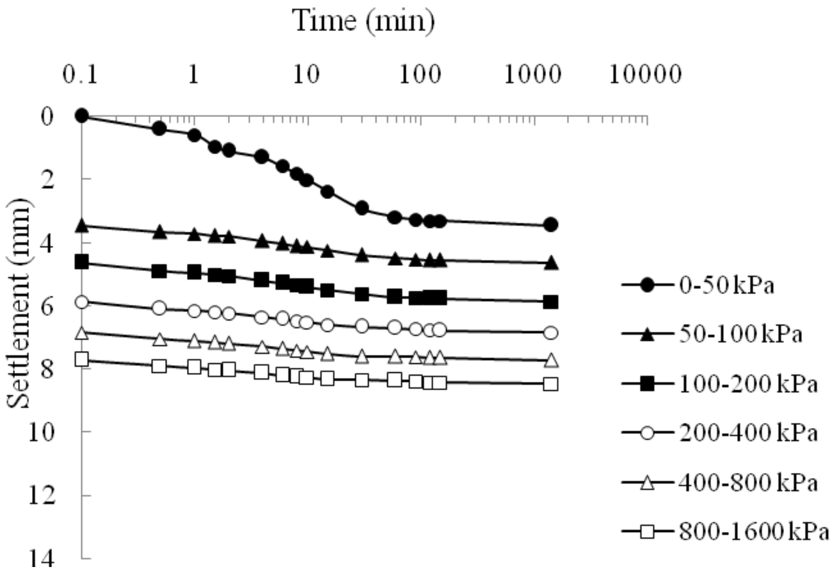

Figure 4.9 Settlement versus *log t* curve for sample with plasticity index 43

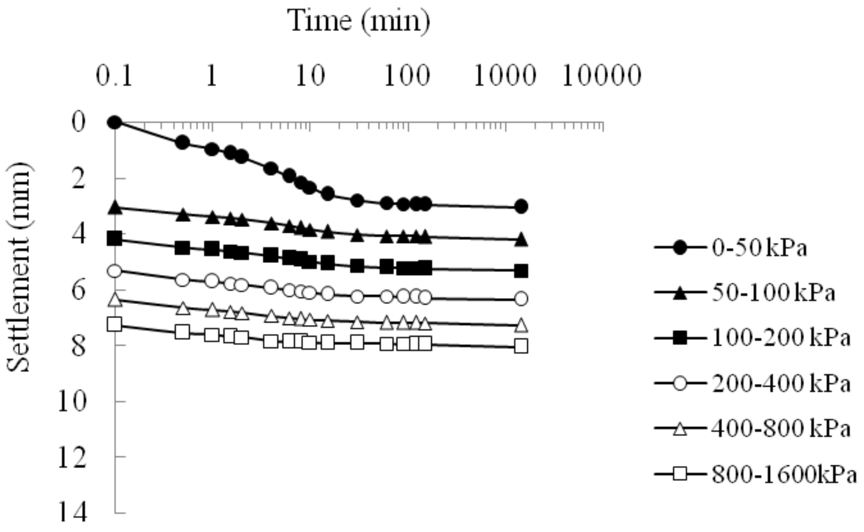

Figure 4.10 Settlement versus *log t* curve for sample with plasticity index 33

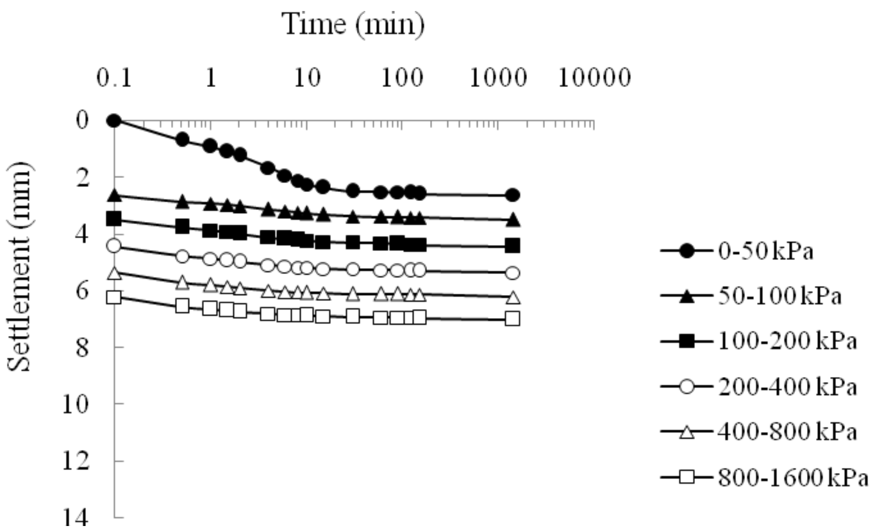

Figure 4.11 Settlement versus *log t* curve for sample with plasticity index 21

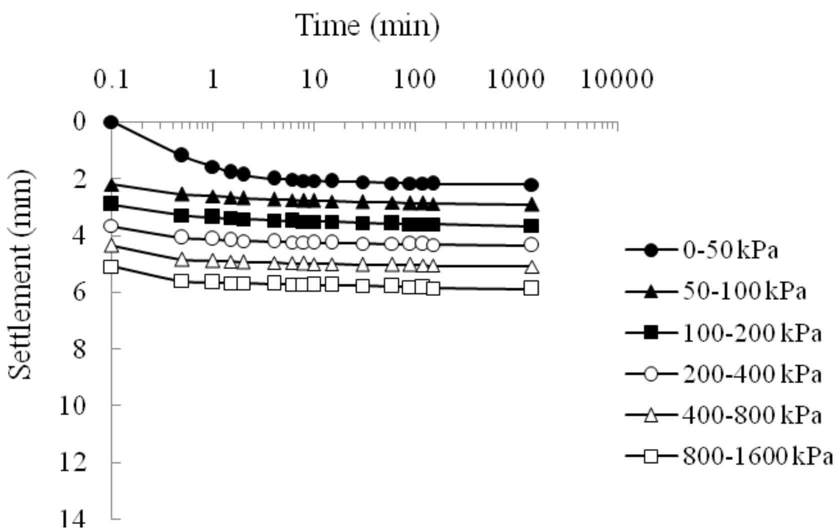

Figure 4.12 Settlement versus *log t* curve for sample with plasticity index 13

Table 4.3 Final settlement corresponding to plasticity index of different soils

Plasticity index (I_p %)	Final settlement (mm) corresponding to stress range 800-1600 (kPa)
102	12.2
78	9.34
53	8.6
45	8.54
43	8.47
33	8.06
25	7.58
21	7.03
17	5.9
13	5.89

4.3 Primary compression index (C_c) for Na and Ca bentonite silt mixture

Primary compression index (C_c) was computed from e versus log σ' graph at the location where primary consolidation ends. The values of C_c for soils with different plasticity index are presented in Table 4.4. The present experimental values of C_c have been compared with those values by using available empirical relations of C_c with I_p, w_l. The same has been shown in Table 4.5. From the Table 4.5 it is observed that with increase in the plasticity of the soil, the C_c value increases. It was also found that the experimental values are in good agreement with values obtained by using equations given by Terzaghi & Peck (1967), Worth and Wood (1978) for remolded clay for low to medium plastic clay (Table 4.5 and Figure 4.15). For highly plastic clay, the difference between observed and the values obtained by using available empirical equations are more. This is true for both Na and Ca bentonite mixes.

4.4 Prediction of C_c from experimental data

From the present experimental data, the correlation was established between C_c and plasticity index (I_p) using logarithmic fit. The empirical relation is shown in equation 1 whose coefficient of regression (R^2) value was found to be 0.950. The same is shown in Figure 4.13. Figure 4.14 shows a comparison between experimental and predicted C_c. The deviation of the experimental data from the predicted data is -16.7% to 7.4% which is considered to be reasonably accurate in such type of problem.

$$C_c = 0.23 \ln\left(I_p\right) - 0.438 \tag{1}$$

Table 4.4 Variation of C_α and C_c for various soils with different I_p and at different stress range

Na bentonite mixed with silt						
Soil	Ratio (Bentonite : Silt)	Plasticity Index (I_p %)	Pressure range(kPa)	C_a	C_c	C_a / C_c
A	50 : 50	102	0-50	0.068	0.68	0.1
			50-100	0.066		0.097
			100-200	0.059		0.086
			200-400	0.055		0.08
			400- 800	0.0533		0.078
			800- 1600	0.0298		0.043
B	40 : 60	78	0-50	0.0478	0.48	0.099
			50-100	0.044		0.091
			100-200	0.04		0.083
			200-400	0.035		0.072
			400- 800	0.025		0.052
			800- 1600	0.015		0.031
C	30 : 70	45	0-50	0.043	0.44	0.1
			50-100	0.037		0.084
			100-200	0.027		0.061
			200-400	0.02		0.045
			400- 800	0.015		0.034
			800- 1600	0.013		0.029
D	20 : 80	25	0-50	0.028	0.28	0.1
			50-100	0.014		0.05
			100-200	0.011		0.039
			200-400	0.009		0.032
			400- 800	0.008		0.028
			800- 1600	0.007		0.028
E	10 : 90	17	0-50	$5.74X10^{-3}$	0.2	0.029
			50-100	$5.74X10^{-3}$		0.029
			100-200	$5.74X10^{-3}$		0.029
			200-400	$5.74X10^{-3}$		0.029
			400- 800	$5.74X10^{-3}$		0.029
			800- 1600	$5.74X10^{-3}$		0.029

Table 4.4 (Continued)

Ca bentonite mixed with silt						
F	50 : 50	53	0-50	0.0246	0.47	0.052
			50-100	0.017		0.036
			100-200	0.015		0.032
			200-400	0.013		0.028
			400- 800	0.012		0.025
			800- 1600	0.011		0.023
G	40 : 60	43	0-50	0.014	0.43	0.044
			50-100	0.0133		0.032
			100-200	0.012		0.03
			200-400	0.011		0.027
			400- 800	0.01		0.025
			800- 1600	0.01		0.023
H	30 : 70	33	0-50	0.01	0.37	0.027
			50-100	0.01		0.027
			100-200	0.01		0.027
			200-400	0.01		0.027
			400- 800	0.01		0.027
			800- 1600	0.01		0.027
I	20 : 80	21	0-50	0.007	0.27	0.025
			50-100	0.007		0.025
			100-200	0.007		0.025
			200-400	0.007		0.025
			400- 800	0.007		0.025
			800- 1600	0.007		0.025
J	10 : 90	13	0-50	0.004	0.16	0.02
			50-100	0.004		0.02
			100-200	0.004		0.02
			200-400	0.004		0.02
			400- 800	0.004		0.02
			800- 1600	0.004		0.02

Table 4.5 Comparison of present experimental C_c values with available empirical relations and predicted from experiment

Ip	$C_c = \Delta e / \Delta log\ \sigma^{l}$ (from present experiment)	Terzaghi and Peck (remolded) clay(1967) $C_c = 0.009(LL-10)$	Wroth and Wood(1978) $C_c = 0.5Gs(PI\%)/100$	C_c predicted From experiment $C_c = 0.23ln(Ip) - 0.438$	Deviation from experiment (%)
13	0.16	0.20	0.16	0.15	6.3
17	0.20	0.21	0.22	0.21	-5.0
21	0.27	0.27	0.27	0.26	3.7
25	0.28	0.28	0.32	0.30	-7.1
33	0.37	0.39	0.43	0.37	0.0
43	0.43	0.49	0.56	0.43	0.0
45	0.44	0.44	0.59	0.44	0.0
53	0.47	0.63	0.7	0.47	0.0
78	0.48	0.68	1.02	0.56	-16.7
102	0.68	0.86	1.35	0.63	7.4

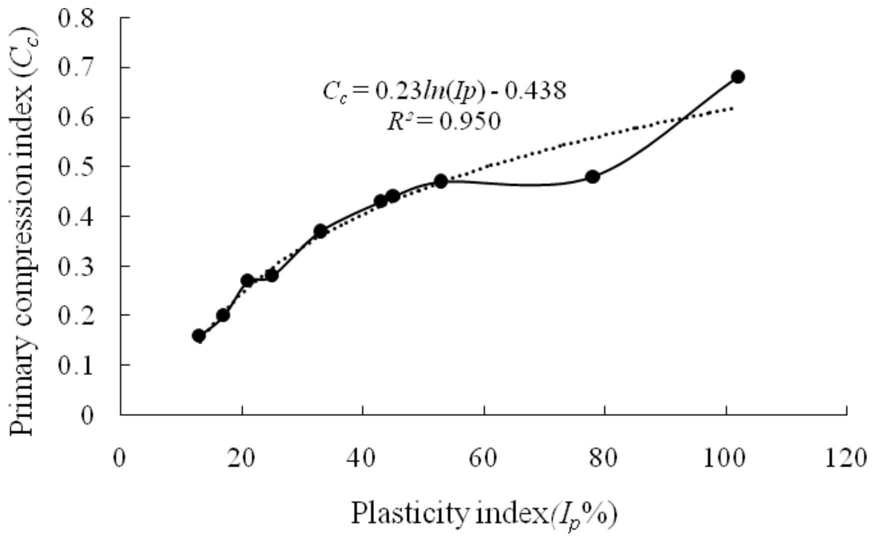

Figure 4.13 C_c versus *Ip* plot for Na and Ca bentonite blended with silt predicted from experiment

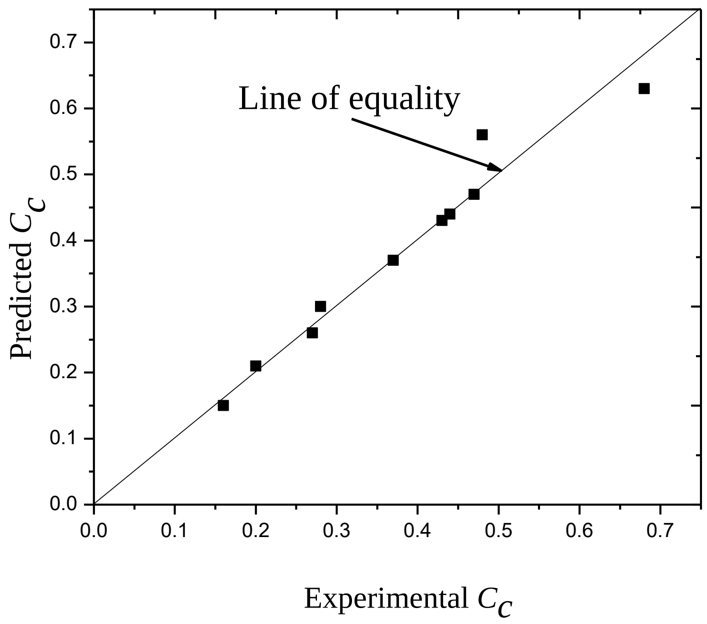

Figure 4.14 Comparison of C_c experimental with Predicted

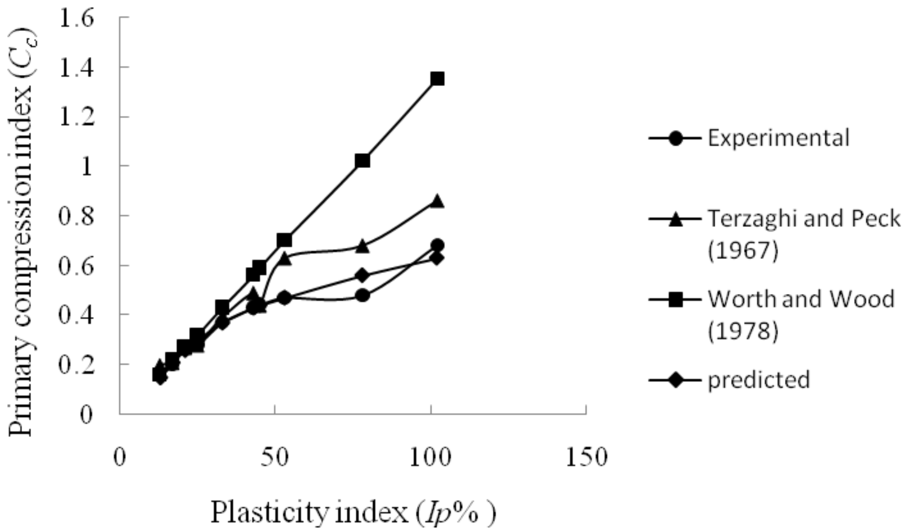

Figure 4.15 Comparison of C_c versus I_p

4.5 Secondary compression index (C_α) for Na and Ca bentonite silt mixture

The values of secondary compression index (C_α) at each consolidation pressure ($\sigma^{|}$) were obtained from the linear segment of the settlement versus *log t* curve immediately beyond the transition from primary to secondary compression. The linear portion of the settlement versus *log t* curve shown in Figures 4.3 through 4.7 (Na-bentonite mixed with silt) and Figure 4.8 through 4.12 (Ca-bentonite mixed with silt) separates the secondary compression region from primary compression for soil of different plasticity index. The ratio of change in void ratio (Δe) to change in logarithm of time ($\Delta\ log\ t$) in the linear region will give secondary compression index (C_α). The variation of C_α with plasticity index (I_p) and stress range is shown in Table 4.4 It is observed that C_α decreases with increase in stress range i.e. $[C_\alpha]_1 > [C_\alpha]_2 > [C_\alpha]_3 > [C_\alpha]_4 > [C_\alpha]_5 > [C_\alpha]_6$ (1, 2, 3, 4, 5, 6 are incremental loading). This is in conformity with the findings given by Mesri and Castro (1987) for natural soils, the secondary consolidation at three different consolidation pressure $[C_\alpha]_1 > [C_\alpha]_2 > [C_\alpha]_3$. The variation of C_α versus plasticity index with respect to stress range is shown in Figures 4.16 and 4.17. From Figures 4.16 and 4.17 it is observed that C_α decreases with increase in stress range but increases with increase in plasticity index. Figures 4.18 and 4.19 show C_α versus $log\ \sigma'$ curves for different I_p. From figures 4.18 and 4.19, it is to be worth noting that for low to medium plastic clays, C_α is independent of consolidation pressure while for highly plastic clays, C_α is dependent on consolidation pressure.

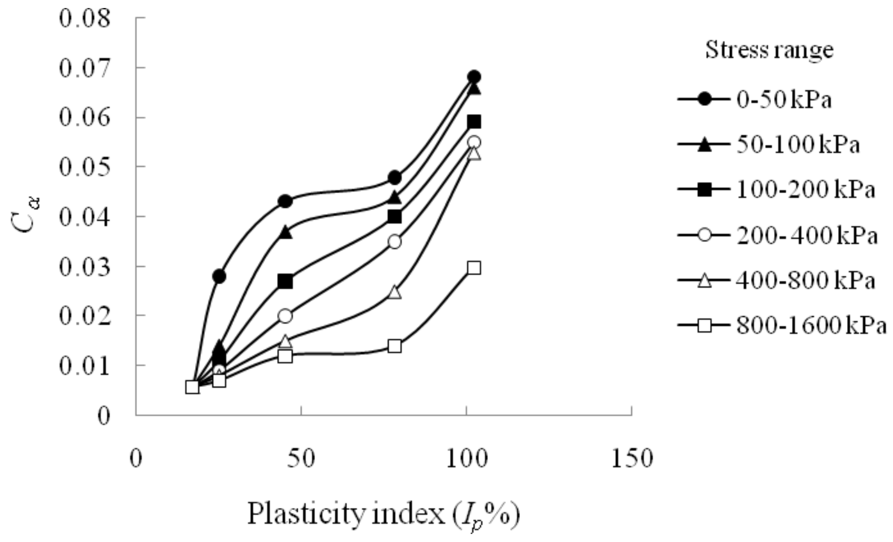

Figure 4.16 C_α versus I_p curve for Na bentonite silt mixes

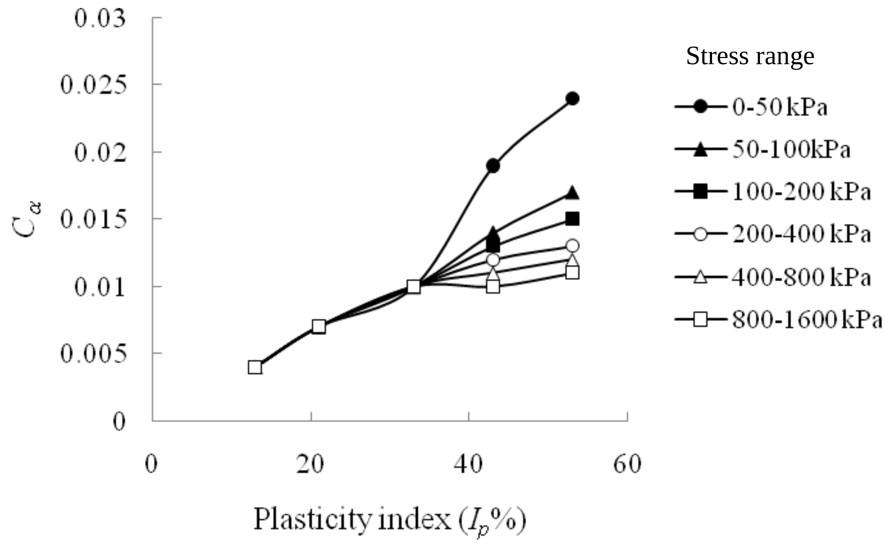

Figure 4.17 C_α versus I_p curve for Ca bentonite silt mixes

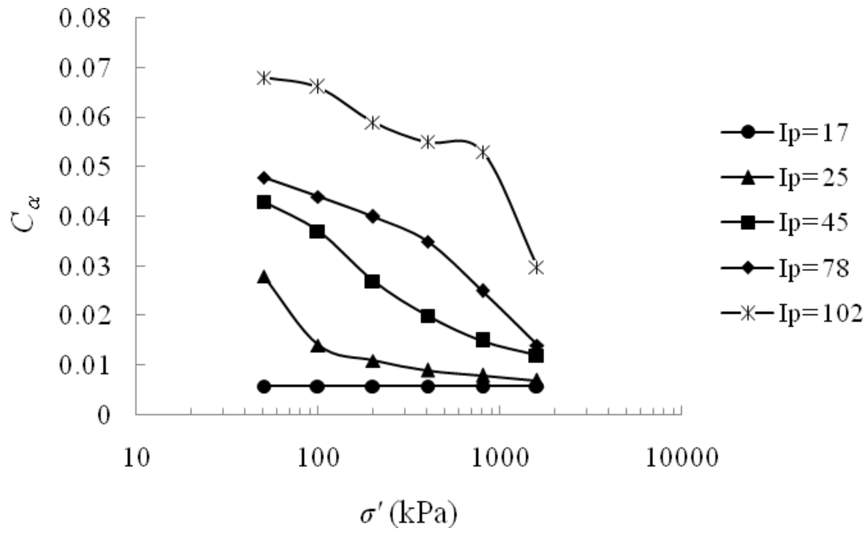

Figure 4.18 C_α versus stress range curve for Na bentonite silt mixes

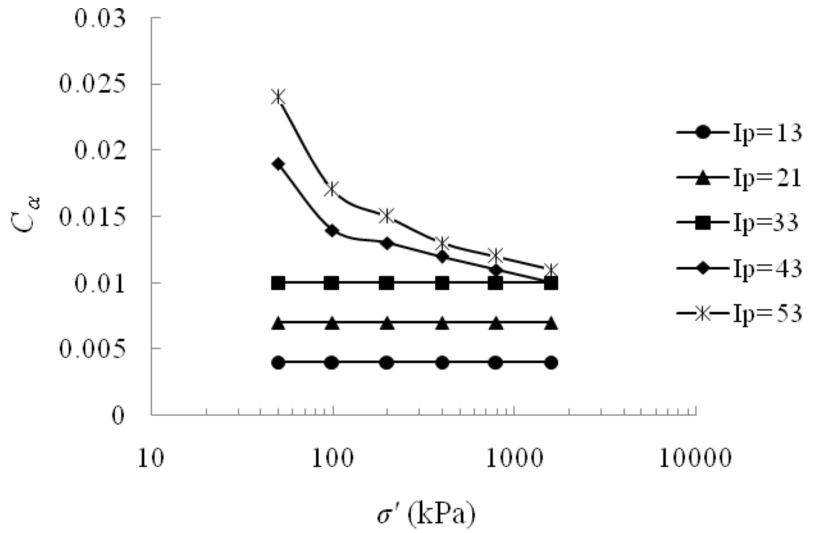

Figure 4.19 C_α versus stress range curve for Ca bentonite silt mixes

4.6 Prediction of C_α from experimental data

The test results were analyzed with Nonlinear Regression Analysis Program (NLREG) software for predicting the correlation of C_α with other parameters. NLREG performs statistical regression analysis to estimate the values of parameters for linear, multivariate, polynomial, logistic, exponential, and general nonlinear functions. The regression analysis determines the values of the coefficients that cause the function to best fit the observed data that is being provided.

Considering C_α to be function of plasticity index (I_p) and stress (σ'), regression analysis were carried out to develop the equation for C_α. The developed equation is

$$C_\alpha = a\left(\frac{I_p}{\log \sigma'}\right) \tag{2}$$

Where

a = 107x 10^{-5}

The coefficient of regression (R^2) value corresponding to this equation is 0.84.

Table 4.6 and Figure 4.20, showcase a comparison between C_α experimental and predicted.

Table 4.6 Comparison between C_α experimental and predicted

Na and Ca bentonite blended with silt				
I_p	σ'	Experimental C_α	Predicted C_α	Deviation from experiment (%)
17	50	0.00574	0.010706	-86.5
17	100	0.00574	0.009095	-58.4
17	200	0.00574	0.007905	-37.7
17	400	0.00574	0.006991	-21.7
17	800	0.00574	0.006266	-9.1
17	1600	0.00574	0.005677	1.0
25	50	0.028	0.015745	43.7
25	100	0.014	0.013375	4.4
25	200	0.011	0.011625	-5.6
25	400	0.009	0.01028	-14.2
25	800	0.008	0.009214	-15.1
25	1600	0.007	0.008349	-19.2
45	50	0.043	0.028341	34.0
45	100	0.037	0.024075	34.9
45	200	0.027	0.020925	22.4
45	400	0.02	0.018505	7.4
45	800	0.015	0.016586	-10.5
45	1600	0.012	0.015028	-25.2
78	50	0.0478	0.049124	-2.7
78	100	0.044	0.04173	5.1
78	200	0.04	0.036271	9.3
78	400	0.035	0.032075	8.3
78	800	0.025	0.028749	-14.9
78	1600	0.014	0.026048	-86.0
102	50	0.068	0.064239	5.5
102	100	0.066	0.05457	17.3
102	200	0.059	0.047431	19.6
102	400	0.055	0.041944	23.7
102	800	0.053	0.037594	29.0
102	1600	0.0298	0.034062	-14.3
13	50	0.004	0.008187	-104.6
13	100	0.004	0.006955	-73.8
13	200	0.004	0.006045	-51.1
13	400	0.004	0.005346	-33.6
13	800	0.004	0.004791	-19.7
13	1600	0.004	0.004341	-8.5

Table 4.6 (Continued)

21	50	0.007	0.013226	-88.9
21	100	0.007	0.011235	-60.5
21	200	0.007	0.009765	-39.5
21	400	0.007	0.008635	-23.3
21	800	0.007	0.00774	-10.5
21	1600	0.007	0.007013	-0.1
33	50	0.01	0.020783	-107.8
33	100	0.01	0.017655	-76.5
33	200	0.01	0.015345	-53.4
33	400	0.01	0.01357	-35.7
33	800	0.01	0.012163	-21.6
33	1600	0.01	0.01102	-10.2
43	50	0.019	0.027081	-42.5
43	100	0.014	0.023005	-64.3
43	200	0.013	0.019995	-53.8
43	400	0.012	0.017682	-47.3
43	800	0.011	0.015849	-44.0
43	1600	0.01	0.01436	-43.5
53	50	0.024	0.033379	-39.0
53	100	0.017	0.028355	-66.7
53	200	0.015	0.024645	-64.3
53	400	0.013	0.021794	-67.6
53	800	0.012	0.019534	-62.7
53	1600	0.011	0.017699	-60.9

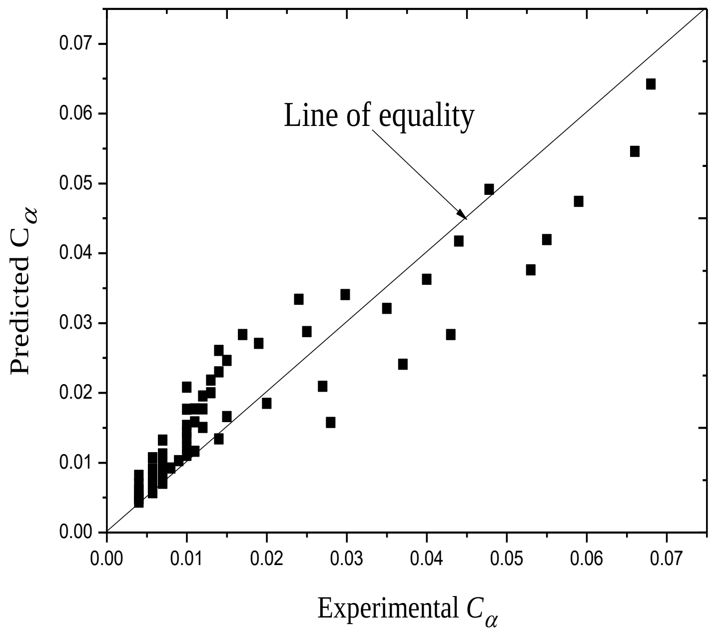

Figure 4.20 Comparison of C_α experimental with predicted

4.7 Prediction of $C_\alpha / (1+e_p)$ from experimental data

From the present experimental data, the correlation was established between $C_\alpha / (1+e_p)$ and plasticity index (I_p) using polynomial fit. The values of e_p corresponding to EOP (end of primary consolidation) for each stress range (from 100 kPa to 1600 kPa) were found out. Seven soils, two from Na bentonite silt mix and five from Ca bentonite silt mix with I_p ranging from 13 to 53% corresponding to soils of *CI* to *CH* (above A-line of Plasticity chart) and *MI* to *MH* (below the A- line of plasticity chart) respectively were considered for analysis. The empirical relation is shown in equation 3 whose coefficient of regression (R^2) value was found to be 0.992. The same is shown in Figure 4.21. Figure 4.22 shows a comparison between experimental and predicted average ($C_\alpha / (1+e_p)$) for each soil. The deviation of the experimental data from the predicted data is -6.14% to 0.8% which is considered to be reasonably accurate in such type of problem. The deviation of predicted ($C_\alpha / (1+e_p)$) values corresponding to all seven soils at stress range from 100 kPa to 1600 kPa from the average predicted (C_α / $(1+e_p)$) values for each soil were calculated. The deviations were in the range of -18% to 24% which may be considered reasonably accurate (Table 4.9).

$$\frac{C_\alpha}{(1+e_p)} = -0.025\,(I_p)^2 + 2.4\,(I_p) - 4.0 \qquad (3)$$

Table 4.7 Experimental values of average (C_α/ $(1+e_p)$) with I_p for each soil

C_α	e_p	$C_\alpha/(1+e_p)$	I_p	Effective vertical stress ($\sigma^{\prime}$)	Avg.($C_\alpha/(1+e_p)$)
0.004	0.96	0.00204	13	100	0.00219
0.004	0.89	0.00212		200	
0.004	0.83	0.00219		400	
0.004	0.78	0.00225		800	
0.004	0.7	0.00235		1600	
0.00574	1.18	0.00263	17	100	0.00281
0.00574	1.11	0.00272		200	
0.00574	1.04	0.00281		400	
0.00574	0.98	0.00290		800	
0.00574	0.91	0.00301		1600	
0.007	1.21	0.00317	21	100	0.00352
0.007	1.11	0.00332		200	
0.007	1.01	0.00348		400	
0.007	0.93	0.00363		800	
0.007	0.74	0.00402		1600	
0.028	1.638	0.00531	25	100	0.00407
0.014	1.522	0.00436		200	
0.011	1.419	0.00372		400	
0.009	1.333	0.00343		800	
0.008	1.25	0.00356		1600	
0.01	1.47	0.00405	33	100	0.00452
0.01	1.34	0.00427		200	
0.01	1.21	0.00452		400	
0.01	1.1	0.00476		800	
0.01	1.01	0.00498		1600	
0.014	1.66	0.00526	43	100	0.00500
0.0133	1.51	0.00530		200	
0.012	1.4	0.00500		400	
0.011	1.27	0.00485		800	
0.01	1.17	0.00461		1600	

Table 4.7 (continued)

0.017	2	0.00567	53	100	0.00504
0.015	1.8	0.00536		200	
0.013	1.7	0.00481		400	
0.012	1.5	0.00480		800	
0.011	1.4	0.00458		1600	

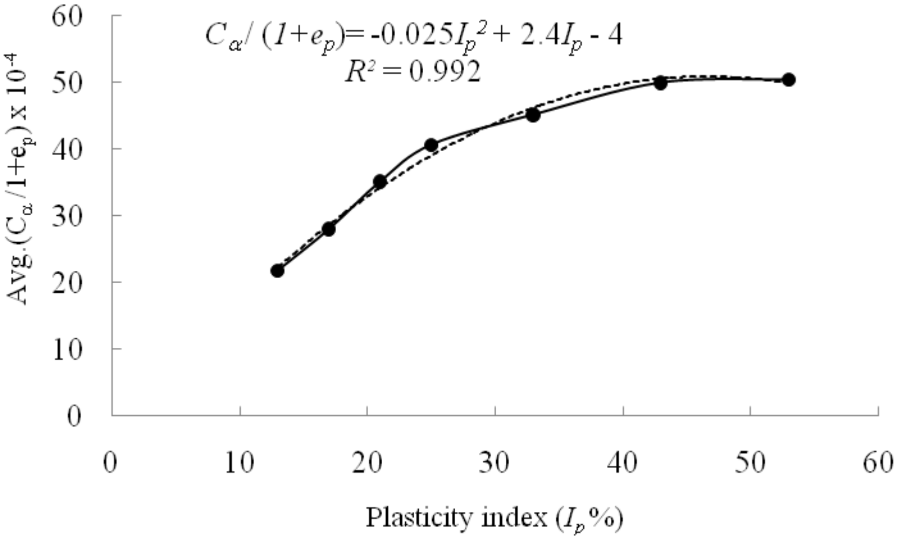

Figure 4.21 C_α / $(1+e_p)$ versus I_p plot for Na and Ca bentonite blended with silt predicted from experiment

Table 4.8 Comparison between avg. C_α / $(1+e_p)$ experimental and predicted

I_p (%)	Experimental Avg. $C_\alpha{}^{\mathrm{l}} = C_\alpha/(1+e_p)$	Predicted avg. $C_\alpha{}^{\mathrm{l}} = C_\alpha /(1+e_p)$	Deviation from experiment (%)
13	21.9	22.975	-4.91
17	28.1	29.575	-5.25
21	35.2	35.375	-0.50
25	40.7	40.375	0.80
33	45.2	47.975	-6.14
43	50	52.975	-5.95
53	50.4	52.975	-5.11

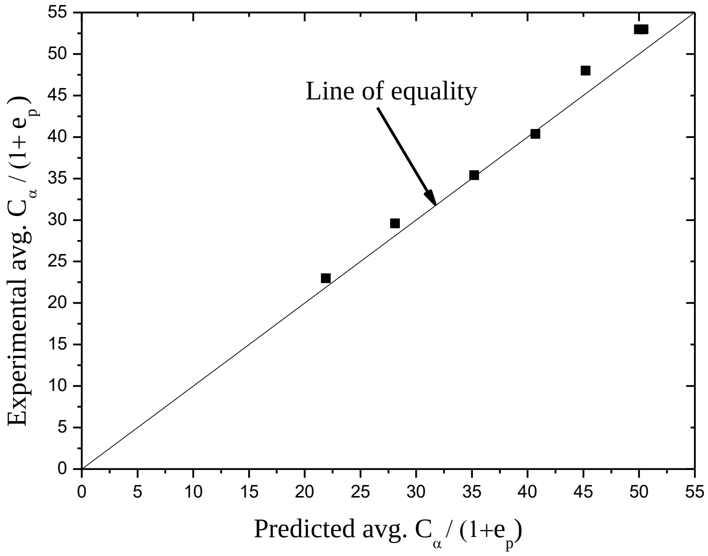

Figure 4.22 Comparison of avg. $C_\alpha / (1+e_p)$ experimental with predicted

Table 4.9 Comparison between $C_\alpha / (1+e_p)$ experimental and predicted with stress range

C_α	e_p	I_p	Effective vertical stress (σ^I)	Experimental $C_\alpha{}^I = C_\alpha /(1+e_p) \times 10^{-4}$	Predicted $C_\alpha{}^I = C_\alpha/(1+e_p)) \times 10^{-4}$	Deviation from experiment (%)
0.004	0.96	13	100	20.4	22.975	-12.6
0.004	0.89		200	21.2	22.975	-8.3
0.004	0.83		400	21.9	22.975	-4.9
0.004	0.78		800	22.5	22.975	-2.1
0.004	0.7		1600	23.5	22.975	2.2
0.00574	1.18	17	100	26.3	29.575	-12.4
0.00574	1.11		200	27.2	29.575	-8.7
0.00574	1.04		400	28.1	29.575	-5.2
0.00574	0.98		800	29	29.575	-1.9
0.00574	0.91		1600	30.1	29.575	1.7
0.007	1.21	21	100	31.7	35.375	-11.5
0.007	1.11		200	33.2	35.375	-6.5
0.007	1.01		400	34.8	35.375	-1.6
0.007	0.93		800	36.3	35.375	2.5
0.007	0.74		1600	40.2	35.375	12.0

Table 4.9 (Continued)

0.028	1.64	0.00531	25	100	53.1	40.375	23.9
0.014	1.52	0.00436		200	43.6	40.375	7.3
0.011	1.42	0.00372		400	37.2	40.375	-8.5
0.009	1.33	0.00343		800	34.3	40.375	-17.7
0.008	1.25	0.00356		1600	35.6	40.375	-13.4
0.01	1.47	0.00405	33	100	40.5	47.975	-18.4
0.01	1.34	0.00427		200	42.7	47.975	-12.3
0.01	1.21	0.00452		400	45.2	47.975	-6.1
0.01	1.1	0.00476		800	47.6	47.975	-0.7
0.01	1.01	0.00498		1600	49.8	47.975	3.6
0.014	1.66	0.00526	43	100	52.6	52.975	-0.7
0.0133	1.51	0.00530		200	53	52.975	0.04
0.012	1.4	0.00500		400	50	52.975	-5.9
0.011	1.27	0.00485		800	48.5	52.975	-9.2
0.01	1.17	0.00461		1600	46.1	52.975	-14.9
0.017	2	0.00567	53	100	56.7	52.975	6.5
0.015	1.8	0.00536		200	53.6	52.975	1.1
0.013	1.7	0.00481		400	48.1	52.975	-10.1
0.012	1.5	0.00480		800	48	52.975	-10.3
0.011	1.4	0.00458		1600	45.8	52.975	-15.6

4.8 Variation of C_α / C_c in Na and Ca- bentonite silt mixed soil

From the above discussion as mentioned in section 4.3 and 4.4 it is clear that C_c is obtained from the slope of the *e-log σ′* at the location where primary compression ends. C_α for each soil is obtained from the linear portion of the secondary compression region of the settlement versus log t curve for various stress range. Table 4.4 depicts that C_c is directly proportional to I_p and C_α / C_c values for all soils with different plasticity (I_p) ranging from 13 to 102 lie within a range of 0.02 to 0.1. This is in conformity with the findings of Mesri and Godlewski (1977) that for a variety of natural soils the values of C_α / C_c are in the very narrow range of 0.025 – 0.10. However, Mesri and Castro (1987) by analyzing huge number of data based on variety of natural materials, including peats, organic silts, highly sensitive clays, shales, as well as granular materials, the values of C_α / C_c are in the remarkably narrow range of 0.02 – 0.10. Figures 4.23 and 4.24 show the plot of present experimental value of C_α / C_c versus I_p for varying stress ranges. It is observed that for any soil of low to high plasticity, C_α / C_c decreases with increase in stress. However, the value of C_α / C_c increases with increase in I_p. This is to be noted that the concept of compressibility (C_α / C_c) as given by Mesri and Godlewski (1977) corresponds to remain within a very narrow range of 0.02 - 0.1. However, in the present investigation, it is observed that the ratio of stress dependency C_α and stress independency C_c for all soils of various plasticity indices are constant i.e. within a very narrow margin of 0.02 to 0.1 for all consolidation pressure. This is true both for Na-bentonite silt mix and Ca-bentonite silt mix.

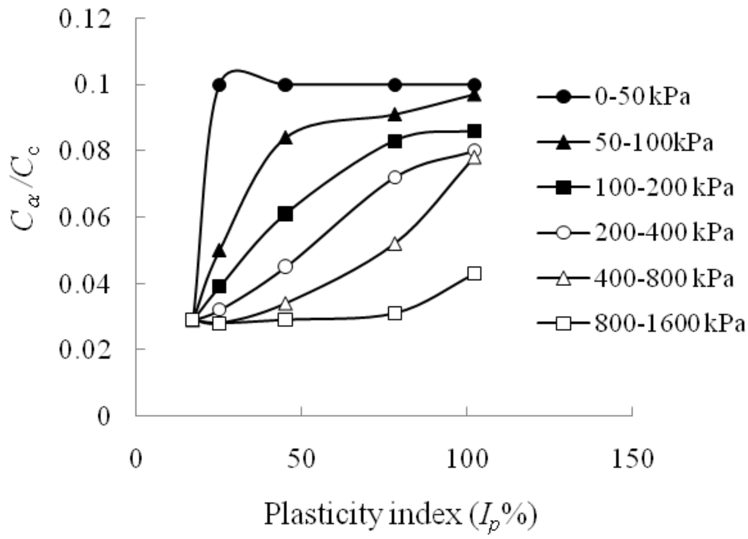

Figure 4.23 C_α / C_c versus I_p curve for Na bentonite silt mixes

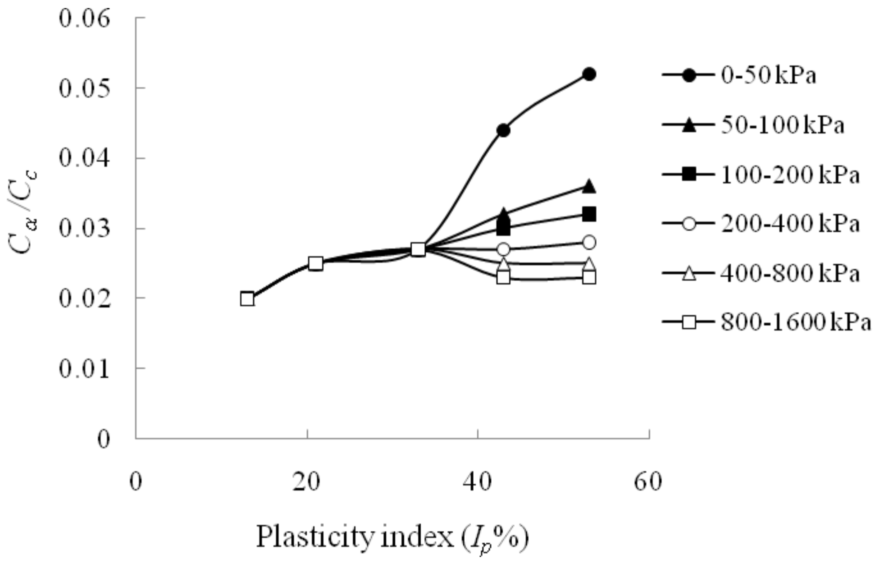

Figure 4.24 C_α / C_c versus I_p curve for Ca bentonite silt mixes

4.9 Variation of C_v, m_v, k in Na and Ca- bentonite silt mixture

C_v, m_v, k are other parameters those govern the consolidation phenomena. C_v defines the time rate of consolidation whereas m_v, the coefficient of volume compressibility which is defined as the change in volume per unit initial volume per unit increase in effective stress. It is interesting to observe that there is not much change in C_v with stress level (Figures 4.25 and 4.26). As the soil becomes more plastic, the C_v is less dependent on stress level. This is noticeable from Figures 4.25 and 4.26.

The plot of m_v with stress level for all soils with different plasticity is shown in Figures 4.27 and Figures 4.28. From Figures 4.27 and 4.28, it is seen that m_v is more stress dependent. The value of m_v is very high for soft clays at low stress level.

From Figures 4.27 and 4.28, it is observed that m_v increases with increase in plasticity index (I_p) and decreases with increase in stress range and merge at higher stress level.

Permeability (k) of soft soil is stress dependent, lower the stress level higher is the permeability (k) as seen from Figure 4.29 and 4.30. When the stress level is very high the permeability of low to extremely high plastic soil are nearly same (Figures 4.29 and 4.30). It is observed from the Figure 4.29 and 4.30 that low permeability is associated with highly soft clay. The permeability (k) versus stress range and I_p is tabulated in Table 4.10.

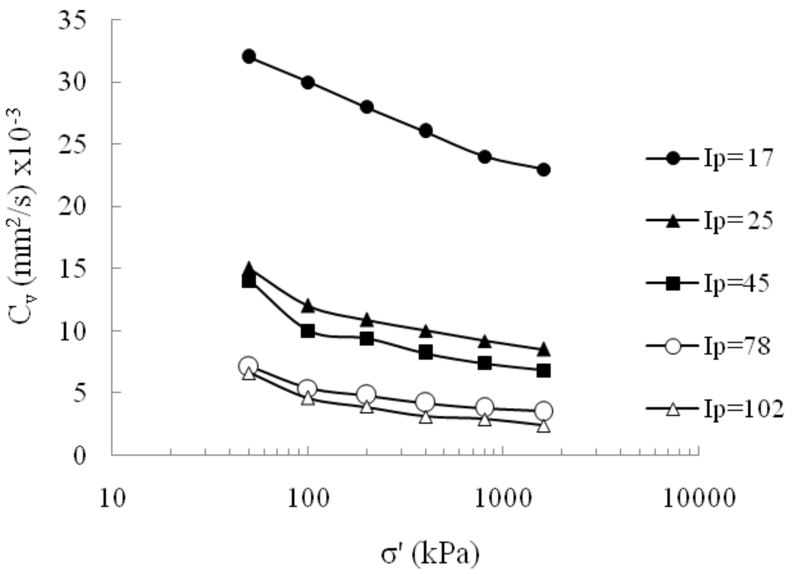

Figure 4.25 C_v versus log σ′ curve for Na bentonite silt mixes

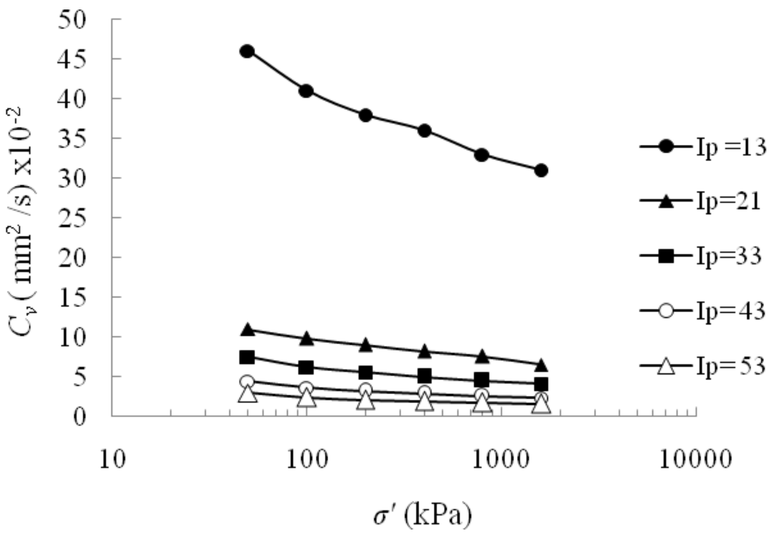

Figure 4.26 C_v versus *log σ′* curve for Ca bentonite silt mixes

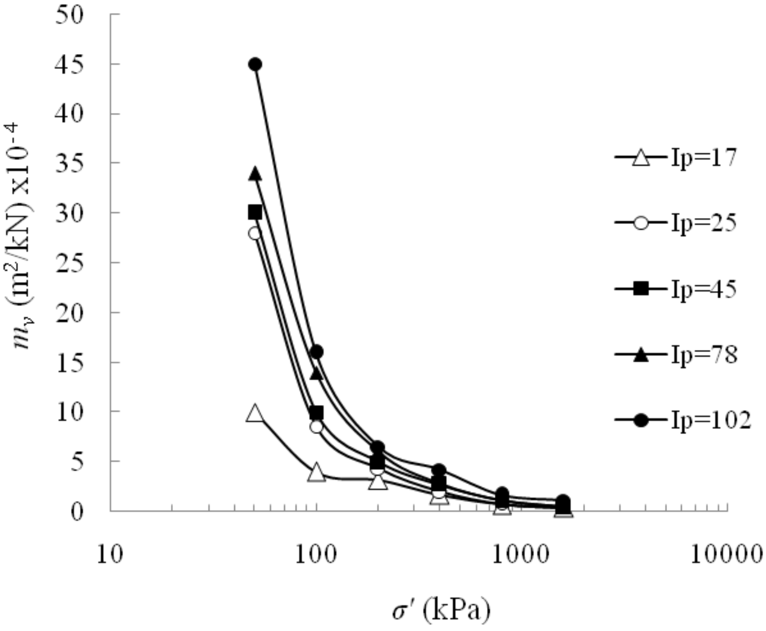

Figure 4.27 m_v versus $log\ \sigma'$ curve for Na bentonite silt mixes

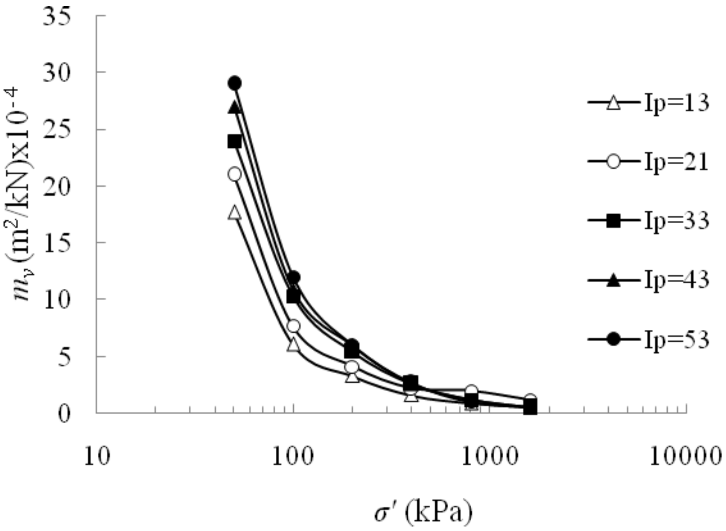

Figure 4.28 m_v versus $log\ \sigma'$ curve for Ca bentonite silt mixes

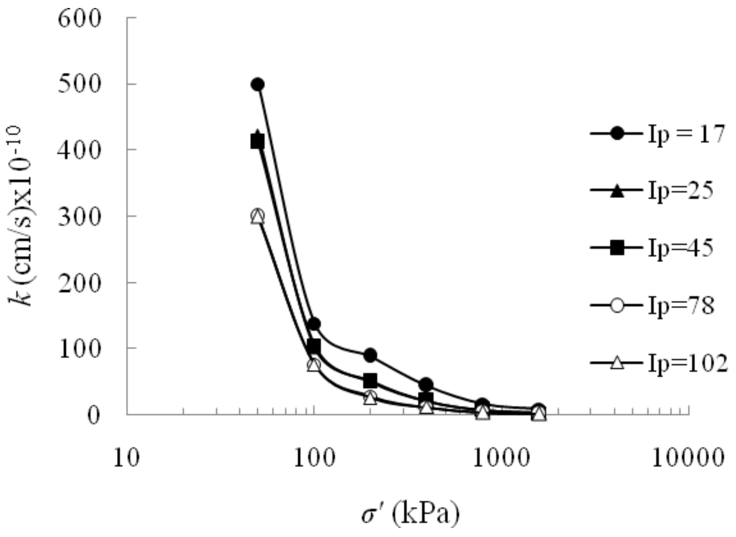

Figure 4.29 k versus $log\ \sigma'$ curve for Na bentonite silt mixes

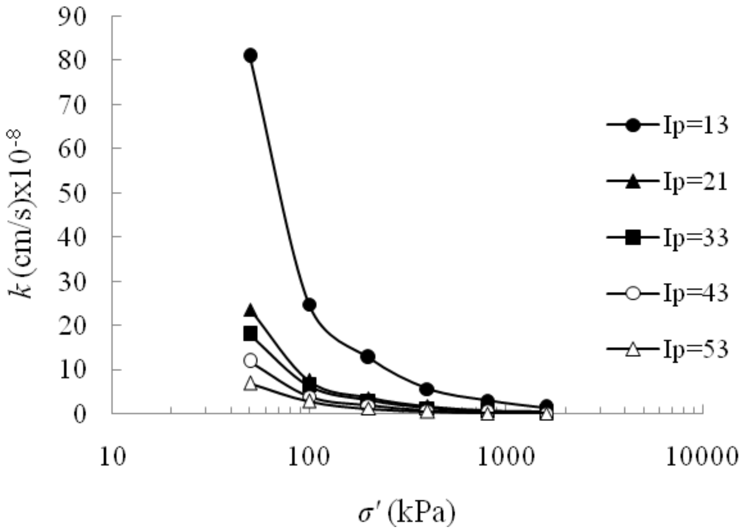

Figure 4.30 k versus $log\ \sigma'$ curve for Ca bentonite silt mixes

Table 4.10 Permeability (k) of soils with different plasticity at various stress range

Soil	Plasticity index (I_p %)	pressure range (kPa)	k (cm / s)
A	102	0 - 50	300×10^{-10}
		50 -100	76×10^{-10}
		100 - 200	26×10^{-10}
		200 - 400	12×10^{-10}
		400 - 800	4×10^{-10}
		800 - 1600	2×10^{-10}
B	78	0 - 50	300×10^{-10}
		50 -100	76×10^{-10}
		100 - 200	28×10^{-10}
		200 - 400	12×10^{-10}
		400 - 800	4×10^{-10}
		800 - 1600	2×10^{-10}
C	45	0 - 50	412×10^{-10}
		50 -100	105.2×10^{-10}
		100 - 200	50×10^{-10}
		200 - 400	21×10^{-10}
		400 - 800	8×10^{-10}
		800 - 1600	3.62×10^{-10}
D	25	0 - 50	423×10^{-10}
		50 -100	102.2×10^{-10}
		100 - 200	51×10^{-10}
		200 - 400	21×10^{-10}
		400 - 800	8×10^{-10}
		800 - 1600	3.62×10^{-10}
E	17	0 - 50	500×10^{-10}
		50 -100	38×10^{-10}
		100 - 200	88.9×10^{-10}
		200 - 400	44.3×10^{-10}
		400 - 800	16.5×10^{-10}
		800 - 1600	9.13×10^{-10}
F	53	0 - 50	7×10^{-8}
		50 -100	2.84×10^{-8}
		100 - 200	1.22×10^{-8}
		200 - 400	0.51×10^{-8}
		400 - 800	0.18×10^{-8}
		800 - 1600	0.077×10^{-8}

Table 4.10 (Continued)

G	43	0 - 50	11.9×10^{-8}
		50 -100	3.86×10^{-8}
		100 - 200	1.89×10^{-8}
		200 - 400	0.722×10^{-8}
		400 - 800	0.3×10^{-8}
		800 - 1600	0.12×10^{-8}
H	33	0 - 50	18×10^{-8}
		50 -100	6.36×10^{-8}
		100 - 200	3×10^{-8}
		200 - 400	1.28×10^{-8}
		400 - 800	0.54×10^{-8}
		800 - 1600	0.22×10^{-8}
I	21	0 - 50	23.7×10^{-8}
		50 -100	7.5×10^{-8}
		100 - 200	3.69×10^{-8}
		200 - 400	1.8×10^{-8}
		400 - 800	0.8×10^{-8}
		800 - 1600	0.65×10^{-8}
J	13	0 - 50	81×10^{-8}
		50 -100	24.8×10^{-8}
		100 - 200	12.8×10^{-8}
		200 - 400	5.7×10^{-8}
		400 - 800	3×10^{-8}
		800 - 1600	1.49×10^{-8}

CHAPTER 5
CONCLUSIONS

5.1 CONCLUSIONS

On the basis of the tests reported in this thesis the following conclusion may be drawn:

- Primary compression index (C_c) increases with increase in the plasticity of the soil.
- Correlation was established between primary compression index (C_c) and plasticity index (I_p). The equation so developed can be considered to be reasonably accurate for consolidation problem.
- Secondary compression index (C_α) decreases with increase in stress range but increases with increase in plasticity index.
- For low to medium plastic clays, secondary compression index (C_α) is independent of consolidation pressure but for highly plastic clay secondary compression index (C_α) is dependent on consolidation pressure.
- Regression analysis was carried out to develop the equation for secondary compression index (C_α). Considering secondary compression index (C_α) to be function of plasticity index (I_p) and stress (σ').
- A correlation was established between effective $C_\alpha/(1+e_p)$ and I_p, which is valid for I_p less or equal to 53%.
- Coefficient of consolidation (c_v) is less dependent on stress for highly plastic soil.
- Coefficient of volume compressibility (m_v) is high for soft clays at low stress level.
- Permeability (k) is inversely proportional to stress level.

5.2 FUTURE SCOPE

- More investigation is required to establish correlation between primary compression index (C_c) and secondary compression index (C_α) with permeability (k), coefficient of consolidation (C_v), coefficient of volume compressibility (m_v).
- Tests may be conducted to study different minerals, pore fluid present in soft soil and their effect on primary and secondary consolidation.
- Oedometer tests may conducted by collecting soils of wide variation of plastic limit from different region and comparison can be made with artificially prepared soil sample in laboratory.
- Tests may be conducted at low and elevated temperature.
- Similar studies may be conducted on natural clay soils. Specifically, natural soils having high organic content are prone to very high secondary compression.

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